UNDRAINED BEHAVIOUR OF SATURATED NORMALLY CONSOLIDATED CLAY UNDER REPEATED LOADING

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CERTIFICATE

Certified that this work has been carried out under my supervision and that this has not been submitted elsewhere for a degree.

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LIST OF SYMBOLS

$\sigma_1', \sigma_2', \sigma_3'$	=	major, intermediate and minor effective stresses
σ_c'	=	effective confining pressure
q	=	deviatoric stress, $(\sigma_1' - \sigma_3')$
p	=	mean effective stress, $(\sigma_1' + 2\sigma_3'/3)$
ϵ	=	strain
U	=	pore water pressure
C_v	=	coefficient of consolidation
n	=	overconsolidation ratio
q_r	=	level of repeated stress
q_{cr}	=	critical level of stress
q_f	=	one-cycle failure stress
U_{eq}	=	equilibirium pore pressure
ϵ_{eq}	=	total strain till equilibirium
N	=	number of stress cycles applied
N_{eq}	=	number of stress cycles required to reach equilibirium

ABSTRACT

This work deals with the undrained behaviour of saturated, normally consolidated, marine clay of high plasticity, subjected to repeated loading. Following aspects have been studied:-

i) the rate of development of pore pressure with respect to strain, during cycling and its relations with the level of cycled stress and effective confining pressure,

ii) the relationship between pore pressure and number of stress cycles applied, and effects of the effective confining pressure and level of cycled stress on this response.

iii) existence of a critical level of stress for repeated loading and its dependence on the effective confining pressure and

iv) post-cycling and post-failure behavioural characteristics.

The pore pressure response of saturated clay under repeated loading may usefully be incorporated in analysis of various soil mechanics problems, where such loading conditions are encountered.

CHAPTER I

INTRODUCTION

Soils in many situations are subjected to repeated loading. Traffic loads, earthquake loads, wind and wave loads and loadings associated with filling and emptying of many engineering structures such as silos and oil tanks are of a repetitive nature.

The particulate nature of soil and presence of water in its pore spaces make the behaviour of soils complex and more susceptible to variations in superimposed loading conditions. Therefore, the response of soils subjected to repeated loading tends to be markedly different from that when subjected to single-cycle loading.

Action of repeated loads results in an essentially undrained shearing of soils. Hence in order to understand the basic behavioural characteristics of soils subjected to repeated loading, it is important to adopt an effective stress analysis. For this a study of pore pressure response under such loading conditions is required.

The number of variables influencing the behaviour of soils subjected to repeated loading are many viz., the stress history, the initial state of stresses, the level

and frequency of repeated stress, the nature of soil (its plasticity etc.), degree of saturation and effective confining pressure.

A number of investigations have been reported on the behaviour of soils under repeated loading. Most of them are without any pore pressure measurement and therefore only a total stress analysis have been used to evaluate the soil behaviour. Only a very few investigations have been reported wherein the effective stress approach has been adopted. Furthermore, there is some controversy regarding the basic nature of pore pressure response under repeated loading, as is obvious from the status of existing literature reviewed in chapter II. Consequently, a general behavioural pattern of soils subjected to repeated loading has not yet emerged.

The object of the present work is to study the response of saturated, resedimented, marine clay of high plasticity under repeated loading with a special reference to the development of pore water pressure. An attempt is made to look into the generality of behavioural trends as observed by other investigators. Among the large number of governing variables as listed earlier, only level of cycled stress and the effective confining pressure

have been chosen for this investigation , because of limited time available. The effect of confining pressure on the general behaviour of high plasticity clay subjected to repeated loading, is for the first time, being reported in this work.

CHAPTER II

A REVIEW OF PREVIOUS SIGNIFICANT STUDIES

2.1 Introduction:

The behaviour of soils subjected to repeated loadings has been the object of field and laboratory studies. Only after 1950 most of experimental studies were made in the laboratory. Almost all of these were made in triaxial testing apparatus. Moreover, only a few studies were accompanied with the measurement of pore fluid pressure in the soil.

Here, the subject has been reviewed in two sections. The first includes only those works in which pore fluid pressure was not measured. And the studies accompanied by measurement of fluid pressure are discussed in the second.

2.2 Repeated Load Studies without Pore Fluid Pressure Measurement:

Kerstern (1943) subjected non-saturated compacted samples of loessial silt and a stabilized sand to static and repeated loading of plate bearing apparatus in the laboratory. He found that subsequent cycles of repeated load produced increasingly larger settlement of the plate which exceeded the total settlement under static load.

Tschebotorioff and McAlpin (1947) tested non-saturated soils with a pluger which was driven into soil samples by a variety of vibrational and slowly repeated loads under both controlled strain and controlled stress. Their findings revealed that a loose uniform sand was particularly susceptible to the action of repeated loads; and that the deformation under a given repeated stress was several times as large as under a similar static load. They noted that deformation was independent of frequency of load application (vibratory or repeated) so long as the vibratory force does not get magnified by resonance. They also found that for low stress levels deformation reached an equilibibirium state after a number of cycles.

Gordon (1959) with upto 500 cycles of load on three different soils concluded that "under repeated cycles of loading, subgrades of fine sandy silts, silty sands and silty clays will deform continuously..... even when the total load is less than that ordinarily considered critical".

Buchnan and Khuri (1954) tested compacted plastic clay in triaxial testing apparatus. They observed that every repeated stress level which exceeded the elastic limit of the soil caused a total collapse of the sample after some number of cycles. For repeated loading below the elastic limit, there was no plastic deformation. They also noted

that the elastic limit increased with the increase of repeated stress level.

Seed, Chan and Monismith (1955) reporting on a series of triaxial tests on compected inactive silty clays under similar repeated axial stress but at rates of load from 1 to 20 cycles per minute, concluded that upto at least 100,000 applications of stress, the specimen deformation depended only on the number of stress applications and was independent of the frequency of stress applications.

Seed and Chan (1957, 1958) found that difference in deformations under different frequencies of stress application occured only due to thixotropic effect. For soils without thixotropic characteristics their previous conclusion was valid. They also observed an increase in strength of their partially saturated samples after the application of repeated stress. This increase in strength was found to decrease with the increase in cyclic stress level. They attributed this stiffening to an internal structural rearrangement of particles rather than to densification.

Seed, McNeill an deGuenin (1958) observed that the stiffening became less pronounced as the degree of saturation of samples approached 100 %. For stiffening they gave a number of possible explainations including changes in moisture

distribution in samples with associated stronger zones, thixotropy and structural rearrangement.

Seed and Chan (1966) found that with respect to deformation a saturated clay behaved quite in the same way as partially saturated soils under repeated loadings. However, they did not find any stiffening effect in case of saturated clay. They also observed similarity in response of anisotropically and isotropically consolidated soils subjected to repeated loadings. Another imporant observation which they made was that cyclic stress with stress reversal (loaded to compresion and extension) caused much larger deformation than a similar stress difference which was all in axial compresion.

Larew and Leonard (1962) based their 'strength criteri for Repeated Loads' on the triaxial compression testing of compacted samples which ranged from very dry to essentially saturated. They concluded that, "For a given soil a critical level of repeated deviotoric stress , q_{cr} exists at which the slope of the deformation versus number of repetitions curve is constant after the first few load applications. For levels of deviatoric stress in excess of this critical value, the deformation curves eventually turn concave upward, their slope continue to increase until failure occurs either by sliding along a shear plane or by bulging. For levels

of stress less than the critical value, the deformation curves eventually approach a horizental asymptote". They proposed that q_{rc} be termed as the strength of soil subjected to repeated loads.

Jhonson (1962), Kwakami and Ogawa (1965) and Ellis(1965) have also reported similar studies. Their findings too are more or less smilar to those of Seed and his associates.

2.3 Repeated Load Studies with Pore Fluid Pressure Considerations:

Bishop and Henkel (1953) performed a series of triaxial compression tests on natural and remoulded Weald clay (Liquid Limit = 43 % ; Plastic Limit = 18 %). They found :-

For a normally consolidated clay in which positive pore fluid pressure is developed during shear, a positive residual pore pressure persists even after the removal of applied shear stress. On reapplication of the same amount of shearing stress a larger pore pressure is developed which is greater in magnitude than would have been developed had the first cycle of load been carried to failure. For a normally consolidated sample which had been stressed beyond failure, on removal of the load the residual pore pressure was observed to attain a value much larger than ever reached during the entire failure cycle.

When a heavily overconsolidated sample was subjected to a shear stress increasingly negative pore pressure were developed after the development of a small positive pore pressure at very low level of stress and strain. When the sample was unloaded from a stress level below failure, a substantial negative pore pressure remained. When the overconsolidated sample was unloaded from a very low level of stress, before negative pore pressure had developed, the residual pore pressure was not in this case negative.

Further working along the same line Henkel (1953) explained the softening of overconsolidated clays. When these clays are subjected to short term stress increase, their behaviour will be essentially undrained and residual negatige pore pressure will exist after unloading. If sufficient time is allowed for this pressure to disipate by intake of water, the overconsolidated clays will soften. He observed that for the case in which drainage is allowed at the end of each cycle, there exists a limiting stress level (less than the drained strength) below which application of repeated stress will not cause much of permanent deformation. But above this level more and more permanent deformation will set in with application of subsequent cycles ultimately causing an exessive deformation. However, he noted that when drainage is allowed at the end of each cycle the

deformation after first cycle is much smaller than would be expected in the softening tests with same load intensity. Moreover, they appear to tend to an upper limit of less than 5 % axial strain.

K.Y. Lo (1961) performed some stress controlled repeated load tests with fluid pressure measurement on clays. His main observations were:-

After each vertical stress increment, the vertical strain progressively increased with a decreasing rate while simultaneously the pore pressure behaved similarly. For unloading, time required to achieve equilibirium for both strain and pore pressurewas shorter. Subsequent cycles took much lesser time. Also amount of variation in strain and pore water pressure decreased with increasing number of cycles.

Rebound curves were non-linear while all recompression curves were linear and parallel to each other. Relationship between pore water pressure and stress difference was non-linear for both loading and unloading. A residual pore pressure persisted corresponding to permanent set in stress-strain curve.

Relationship between pore water pressure and axial strain revealed that both loading and unloading equilibirum

points as well as points during one stress increment/ decrement were observed to lie on the same curve. That is, he found a unique time independent relationship between pore fluid pressure and strain irrespective of direction of stress path, the number of cycles repeated and the position of each load cycle.

Knight and Blight (1965) performed controlled strain repeated triaxial compression tests on normally consolidated and heavily overconsolidated soils. For the normally consolidated clay, they found that pore pressure on removal of stress was greater or less then the original pressure which was obtained during the end of first loading cycles. The peak pore pressures (end of loading) and the residual pore pressures(end of unloading) were found to increase linearly with number of stress applications. They also claimed that it was possible to build up the pore fluid pressure until shear failure.

For the overconsolidated clay they found that the cumulative effect of repeated loading is to build up an increasingly negative pore pressure. At some of levels of repeated stress, this residual negative pore pressure rapidly approached asymptotically a value more negative than the pore pressure in a single cycle failure test.

D.A. Sangrey (1968) made an extensive study of repeated load response of undisturbed Newfield clay. He observed a variety of behaviour patterns which depended upon the level of repeated stress and the number of cycles applied. The most peculiar thing with his clay was that upon removal of deviatoric stress at failure or a little below failure the residual pore pressure was even higher than that obtained at the end of previous loading cycle. The magnitude and direction of residual pore pressure change were found to depend upon the previous consolidation history of the sample and the level of the initial deviatoric stress. In general higher the overconsolidation ratio of the sample the less positive pore pressure would be, if other factors are similar. Likewise , higher the level of initial deviatoric stress the greater the residual strain and pore pressure.

Sangrey found following different behaviour patterns:-

For cycling at a low level of stress difference a limit to the increase of residual strain and pore water pressures developed usually within first ten cycls of loading. Once this limit conditions had been achieved the repeated loading resulted in completely reversible changes in pore water pressure and sample deformation. The sample was said to have reached non-failure equilibirium and its behaviour was essentialy elastic . Upto a certain level of repeated

stress difference such equilibirum condition was always achieved. However, above this critical level pore pressure built up with each successive cycle just as it had in the nonfailure equilibirium tests. But instead of achieving a non-failure equilibirium, the build up of pore pressure continued until the stress obliquity at the peak of loading was equal to the failure stress obliquity for the soil. Each cycle of loading above the critical stress level, q_{cr} resulted in an increase in the residual strain of the sample. The rate of deformation per cycle of load was fairly constent as the pore pressure was building up till the failure and for a number of cycles after the failure was reached. Eventually, however, there was a very pronounced increase in the rate of deformation per cycle and within a few cycles. Thereafter, the sample was unable to support the cycled stress difference leading to an abrupt failure.

When a specimen was loaded repeatedly to the maximum stress difference which it could support, the failure stress difference was successively reduced with each cycle. The failure obliquity was more or less constant so the pore pressure was progressively higher with each cycle eventually a limit peak streangth and failure pore pressure was achieved. At this limit for the cycled-at-failure tests, the pore water pressure did not vary significantly as the sample was

loaded and unloaded. This was probably due, at least in part, to the development of a well defined failure plane in the sample. This limit behaviour was observed for all the cycled-at-failure tests, regardless of the original consolidation history. Moreover an idential ratio of the change in pore water pressure at failure to the deviatoric stress at failure, $\Delta U_f / \Delta(\sigma_1 - \sigma_3)_f$ was observed for all of them.

Sangrey (1968) also gave the concept of equilibirium line. The state of stresses for all the non-failure equilibirium stage for a particular consolidation history when plotted in stress space, define an equilibirium line. The equilibirium line represents the limit to which the effective stresses of an undrained sample will change given a particular initial consolidation and with a particular maximum cycled stress difference. The intersection of an equilibirium line with failure obliquity line defines the critical stress devatoric stress, q_{rc} to which a soil can be subjected to load repetition and still achieve non-failure equilibirium. The inclination of equilibirium line was found to depend on the consolidation history of the sample.

Sangrey further observed a pronounced decrease in the peak strength of a saturated soil after it has been

subjected to a series of repeated loadings. When compared after the same number of loading cycles the peak strength is inversely related to level of cycle stress. When the level of repeated stress is above the critical level, the peak strength continues to decrease with each load cycle.

2.4 Summary:

The behaviour of soils subjected to repeated load in different from the behaviour under a static single load application.

The deformation of a soil under repeated loading is progressive in nature and is greater than produced by the same magnitude of singly applied load. This deformation, due to larger repeated stress, in many cases becomes excessiv and leads to a total collapse. However, the deformation under smaller repeated stress, is observed to attain an equilibirium after some cycles.

It has been observed that repeated loading of some soils with low degree of saturation, as low stress levels, increase their strength. The opposite behaviour of strength decrease due to repeated loading has also been observed.

The deformation of soils without thixotropic characteristics is found to be independent of frequency of repeated loading.

The pore pressure response of soils under repeated loading is also different from that under a static load. It has been shown that after applying and removing of a shear stress, there persists a residual pressure different from the original pore pressure prior to load application. This pore pressure peak as well as residual has been noted to change with subsequent load cycles. In some cases this is progressive until there is an effective stress failure. While in other instances, the pore pressure seems to achieve a limit after a few cycles of stress application. Both normally consolidated and overconsolidated soils show this characteristics of a residual pore pressure. In normally consolidated soils this pore pressure is positive, while in heavily overconsolidated samples there is a negative residual pore pressure.

CHAPTER III

SOIL PROPERTIES AND EXPERIMENTAL SET UP

3.1 Soil Properties:

The soil used in the present investigation was obtained from the Little Rann of Kutch, a Recent deposition in marine environment. The soil is a highly plastic, normally consolidated Illitic clay with some organic constituents.

The classification properties of the soil are:-

Liquid Limit = 91

Plastic Limit = 42

Plasticity Index = 49

Activity (I_p/% clay) = 0.58

Specific Gravity = 2.71

Organic Matter = 11.5 %

Particle size distribution curve is presented in Fig. 3.1. The clay content forms 84 % , rest being silt.

3.2 Sample Preparation:

The soil brought from the field was thoroughly remoulded, mixing it with sufficient amount of water to make a slurry. The slurry was poured into a thick galss tube (6 inches long and 1.45 inches dia) standing vertically over filter paper covered porous stone placed in a tray

filled with water. A porous stone disc (1.4" Dia), with its bottom covered with filter paper was placed on the top of soil in the tube. The soil thus enclosed in a tube, with drainage at the top at bottom was subjected to the following loading scheme to get sufficiently stiff samples:-

Load	Time
0 lb	24 hrs
0.5 "	" "
1.0 "	" "
2.0 "	" "
4.0 "	" "
* 5.0 "	" "
10.0 "	" "

* The samples to be tested under 5 psi effective confinement pressure was loaded up to this stage only.

The soil thus consolidated in the tube was ready to be taken out and set over the triaxial base after being cut into an appropriate length (3 inches). Identical samples were thus prepared from the same slurry following the same loading scheme for consolidation.

3.3 Experimental Set Up:

The triaxial apparatus with cylindrical compression system is the most common device to be employed for study

of soil behaviour in the laboratory. The principal advantages of the triaxial apparatus are the control of drianage, the flexibility of stress system and the measurement of pore fluid pressures. Bishop and Henkel (1962) have presented a very thorough description of the principles and equipments of the triaxial test. Therefore, only a very brief discussion of these will be presented here.

(A) General Lay-Out

A schematic diagram of the equipment arrangement used for the undrained triaxial testing of saturated soils with pore pressure measurement is presented in Fig. 3.2. The three principal parts of this lay out are the triaxial cell and loading frame, the cell pressure system and the pore pressure measuring system.

The soil sample is placed on the base of the triaxial cell and is confined with water which is subjected to a desired pressure obtained from the cell pressure system. The triaxial assembly thus rests on the platform of the loading frame which can be moved up and down at many desired rates. The movement of the platform results in strain controlled loading/unloading of the soil sample as the loading ram resting on the sample top,butts against the bottom of a proving ring hanging from a fixed beam on the loading frame.

The applied axial load is being measured through the deformation of a proving ring. And, the deformation of the sample is measured from a dial gauge (L.C. = .00001 inch) fixed between the proving ring and the top of the cell. The pore pressure developed in the sample is measured from the manometer/gauge of the pore pressure system.

(B) Triaxial Cell

Triaxial cell assembly has following principal features:-

i) The Base:- It is a machined piece of stainless steel. It has got a pedestal in its centre over which a porous stone disc is kept forming a seat for the soil sample. Provisions for four pore pressure, volume change and cell pressure connexions are available. All of them terminate in separate push-pull valves around the periphery of the base through which proper connexion can be made with polythene tubings. Two of them lead from the base of the sample, one is employed for connexion to the pore pressure measuring system while another to the volume-change measuring device. The third valve may be employed to take a lead from the loading cap and hence the top of the sample. And the fourth valve lead to the top of triaxial base and is connected to cell pressure system.

ii) The Removable Cylinder and Top Cap:- A transparent Perspex cylinder is used, which facilitates the setting up of the test and enables the mode of failure to be observed. Strain measurements can also be made optically. The cylinder is permanently fixed to the top cap with a central boss which forms the bush through which a stainless steel ram slides. There are two holes in the top cap which are used for releasing air and filling oil in the cylinder. A short pillar to carry the arm for the axial strain dial gauge is also fixed on the top cap. There is a collar attached to the bottom of the cell with the help of four tie bars. The only joint to be made for setting up the assembly is between the lower collar and O-ring set in the base of the cell. For this hand tightening of four wing nuts is sufficient.

iii) The Loading Ram: It consists of a ground stainless steel rod running in a bush lubricated with oil.

iv) The Loading Cap: A thick metallic disc with a central groove is used to transmit the load from the ram to the sample. The ram gets into the groove and butts against a small steel ball placed in it.

v) Rubber Membrane: The sample is enclosed in thin rubber membranes, 5 to 6 inches in length. Two rubber membranes are sealed against the smooth surfaces of the

loading cap and the pedestal by rubber O-rings under tension, sprung into place from the end of a metal tube.

vi) Side-drains: Besides providing drainage at the top and bottom of the soil sample, side drains are also used to:

a) reduce the time required for full dissipation of pore pressure during consolidation stage.

b) accelarate the equalization of pore pressure within the specimen during undrained loading.

The side drains consist of six filter paper strips (3¼ inches long and ¼ inch wide) put around the sample with their ends folded and attached to the top and bottom filter papers.

(C) Loading System:

Strain controlled axial load is applied with the help of a screw jack operated by an electric motor and gear box. The platform, on which the cell rests, is raised by a screw. The rotating nut which drives the screw is operated through a gear by an electric motor. For rapid adjustment the key is withdrawn from the key-way on the screw and then rotated by a knurled collar. The different rates of strain are obtained by different basic speeds of geared induction motor and alternative pairs of fixed-centre change wheels.

(D) The Cell Pressure System:

To achieve, with sufficient accuracy a constant cell pressure over a long period the self-compensating mercury system is used. The pressure of water in the triaxial cell results from the difference in level between the mercury surfaces in two small cylinders, connected by a thin flexible tube which forms, in effect, the two limbs of a manometer. One of the cylinders rests below, while another hangs through a spring from a carriage which can be moved on a rail fixed on the wall. The height of the upper cylinder can be adjusted to achieve a required cell pressure.

The maintenance of a constant cell pressure is due to self-compensation in mercury level in the upper cylinder. When the cell pressure drops, a little of mercury drains out from the upper cylinder resulting in a loss of difference in elevations of mercury levels in two cylinders. But as the weight of upper cylinder is also reduced correspondingly, the spring shortens raising the top cylinder causing an increase in the elevation of mercury level. The spring is so designed that loss in elevation through drainage of mercury is equal to gain in elevation through shortening of the spring Thus the original difference in elevation of mercury levels in two cylinders is maintained constant and hence the cell pressure is maintained at a constant original value.

With one pair of mercury cylinders the maximum cell pressure is limited because of the limited space available for the movement of the upper cylinder. To get an additional cell pressure, if required, an extra pair of mercury cylinders is used. The low cylinder again rests on the same level while upper one hangs through a spring from a fixed level. The variable pressure from the first pair is taken to the top of upper mercury cylinder of the second pair. And finally, the combined pressure from both the pairs, which can be varied, is taken from the bottom cylinder of second pair to the cell.

The cell pressure is measured from the pressure gauge mounted on a board. A cylinder is also mounted on the same board and connected to the limb which houses all the valves to operate the cell pressure system.

(E) The Pore Pressure Measuring System:

A null method of pressure measurement is used with advantage to measure pore fluid pressure. Three principal parts of the pore pressure measuring system are:-

i) Null Indicator

ii) A Cylinder along with a tube housing valves

iii) A Manometer and a Gauge

The null indicator consists of a fine - bore U - tube in a Perspex block. Two limibs have their bottoms immersed

in a mercury pot. Their tops come out of the perspex block and are connected through thin bore copper tubes, right one to the sample base and left to the cylinder with a piston mounted on the same board. An increase in pore water pressure in the sample will tend to decrease the mercury in the right limb of the U - tube. This is immediately balanced by adjusting the piston in the cylinder to increase the pressure in the left limb by an equal amount, which can be measured either from the bourdon tube pressure gauge or the manometer depending upon its magnitude.

(F) Back Pressure

To ensure complete saturation, to avoid any negative pore fluid pressure and to dissolve the air between the rubber membranes and the sample, a back pressure is applied within the sample.

Here in this study a constant back pressure of 30 psi was used in all the tests. The back pressure which is again obtained from a self - compensating mercury column was applied after consolidation and during shear.

3.4 An Outline of Testing Procedure:

After dealing with each important feature of the lay-out for the triaxial test separately, an integrated out line of the testing procedure is given for a consolidated

undrained test.

i) The soil sample is prepared in the tube through a stage - wise consolidation scheme.

ii) The sample, taken out of the tube, cut to size and weighed, is set on the pedestal of the base. For drainage control, a porous stone with filter paper at its top and a filter paper are provided at the bottom and the top of the soil sample. The rubber membranes are slid over the soil sample and sealed against the pedestal and loading cap to perfectly enclose the sample. The triaxial set is assembled and the cell is filled with water and a little oil at the top to avoid leakage and to provide lubrication.

iii) All connexions are made, the base of the sample is connected to a burette and the base of the cell to the cell pressure system.

iv) The cell pressure is applied and the sample is left to consolidate for 24 hours. Time versus volume change (from burette) readings are taken. When the required consolidation pressure is high this consolidation is performed in stages.

v) After consolidation is complete, the base of the sample is connected to the back pressure system through the pore pressure system to get a desired back pressure. The cell

pressure is also increased correspondingly to get the same effective confining pressure. The sample is left under this back pressured condition for another 12 hours.

vi) The contact of loading ram is made with the ball in the groove of the loading cap. The sample is loaded at a constant strain rate. The readings from the proving ring dial gauge, the strain dial gauge and pore pressure manometer/ gauge are noted at suitable intervals.

vii) The sample is loaded to failure or to a desired pre-failure stage and then unloaded at the same strain rate. A number of pre-failure or post-failure cycles of load are applied.

viii) At the end of the test, the cell is isolated and the sample is unloaded. The triaxial assembly is dismantled. The sample is weighed and small specimen is taken from the centre of the sample for water content determination.

3.5 Rate of Shear:

When the sample is sheared under undrained condition, the pore pressure developed is not uniform throughout the sample. Based on the coefficient of consolidation, C_v (10^{-4} in^2/min)a rate of strain (0.016 % per minute) is worked out to allow 95 % desipation of pore pressure difference within the sample.

3.6 **Area Corrections:**

To calculate stresses at any stage of loading the C/S area of the sample is corrected for changes during consolidation and shearing.

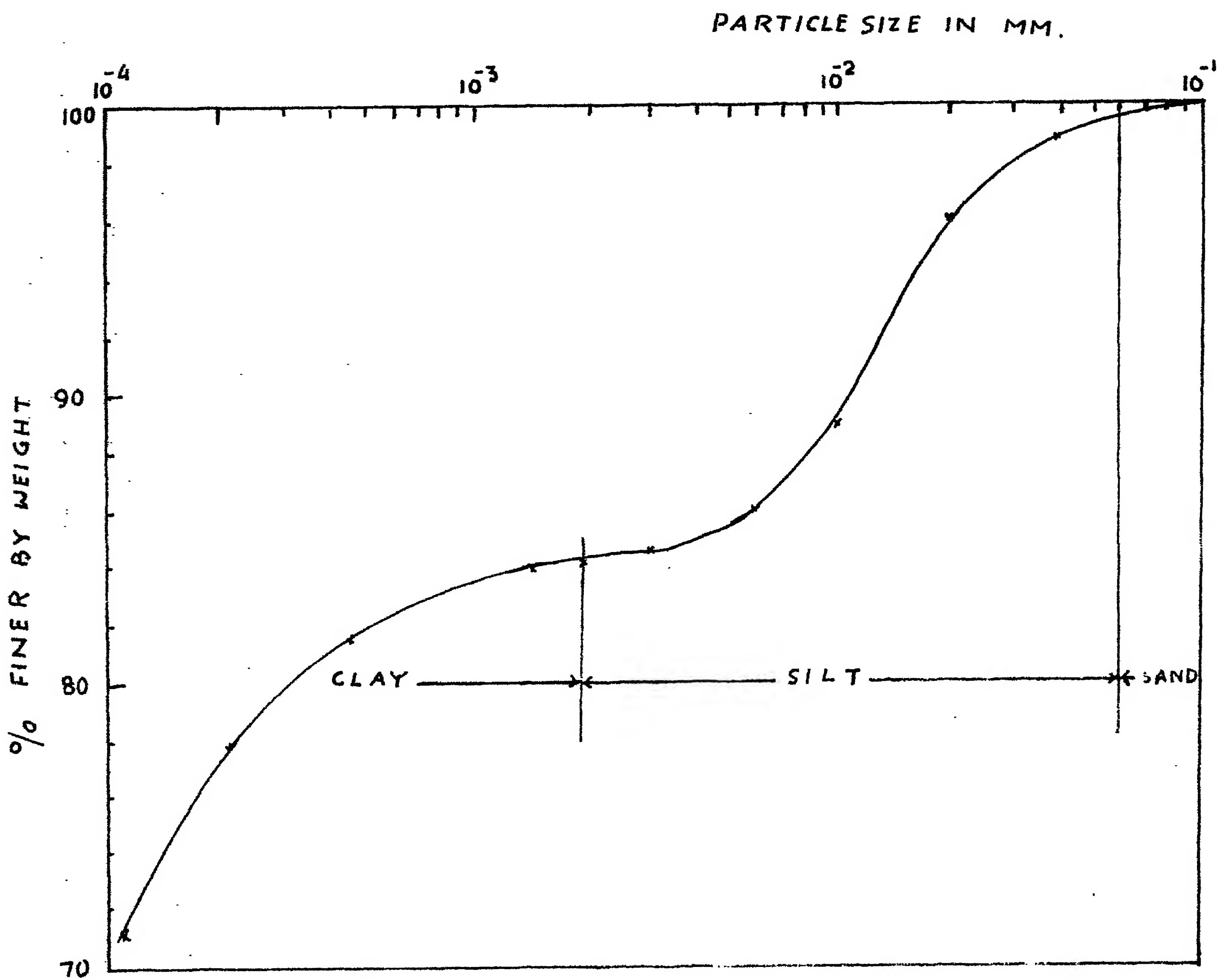


Fig 3-1 : Particle Size Distribution.

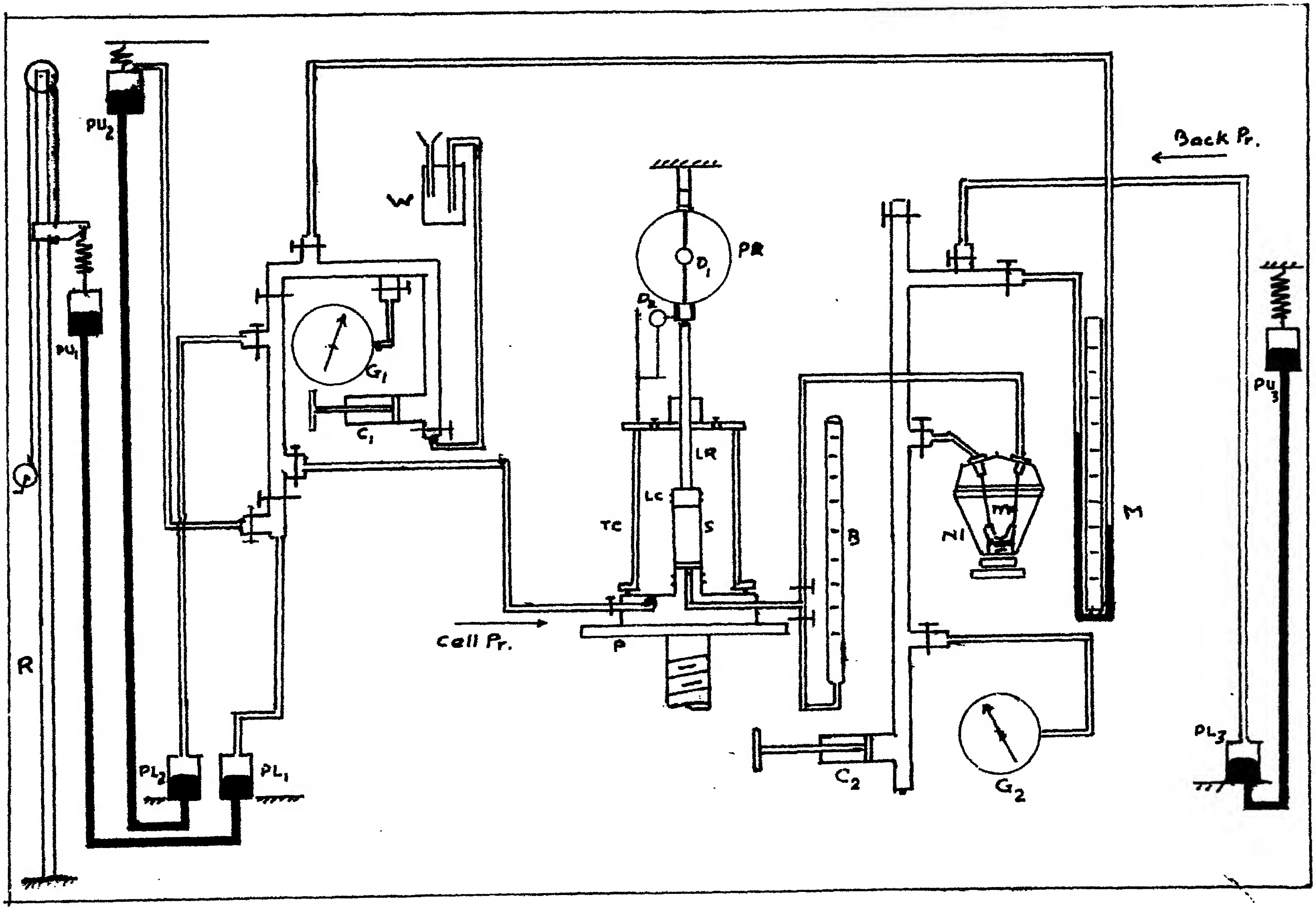


Fig. 3-2. General Lay-Out Of The Experimental Set Up.

Symbols Used in Fig. 3-2 :

PU_1, PL_1	=	Pair Of Mercury Cylinders Used For Cell Pressure (Variable)
PU_2, PL_2	=	" " " " " (Fixed)
PU_3, PL_3	=	Pair of Mercury Cylinders Used For Back Pressure
G_1	=	Gauge to Measure Cell Pressure
C_1	=	Cylinder To Take water in the Cell Pr System.
TC	=	Triaxial Cell
LR	=	Loading Ram
LC	=	Loading Cap
S	=	Sample
P	=	Plat Form Of Loading System
PR	=	Proving Ring
D_1	=	Dial gauge To Measure Load
D_2	=	" " To Measure Strain
B	=	Burette To Measure Volume Change
NI	=	Null Indicator Of Pore Pr. Measuring System
m	=	Manometer of 'NI' (Fine - Bore)
M	=	Manometer To Read Pore Pr
G_2	=	Gauge To Measure Pore Pr

CHAPTER IV

PRESENTATION OF TEST RESULTS

4.1 Introduction:

In this chapter the results obtained from the tests series are being presented. Stress-strain relationship, pore water pressure-strain response and the stress-path followed in stress-space during each test are presented.

Three test series were conducted each for a different effective confining pressure. A series contains three or four independent undrained triaxial tests, with different levels of repeated stress. They are:-

Series A: Normally consolidated at $\sigma_c' = 5$ psi

Test No.	Level of cycled stress, q_r (psi)
TA-1	Failure stress $q_f = 4.8$
TA-2	1.74
TA-3	2.55

Series B: Normally consolidated at $\sigma_c' = 30$ psi

Test No.	Level of cycled stress, q_r (psi)
TB-1	Failure stress $q_f = 19.2$
TB-2	4.42
TB-3	9.00
TB-4	14.20

Series C: Normally consolidated at σ_c' = 45 psi

Test No.	Level of cycled stress, q_r (psi)
TC-1	Failure stress q_f = 28.3
TC-2	7.24
TC-3	14.74

As the test results from different series but corresponding to comparable level of cycled stress(ie. having similar q_r/q_f) are more or less alike, only the results of series A tests are presented in detail. The results of other tests are also presented separately but only new features which are not observed in series A tests are dealt in detail.

On all the figures (Figs. 4-1 to 4-20), the curves for respective single-cycle loading are shown with chain lines and extensions (if any) of the actual curves are shown with dotted lines.

4.2 Series A: Normally consolidated at σ_c' = 5 psi

Test TA-1:

The sample was consolidated isotropically at an effective confining pressure of 5 psi. It was then subjected to axial undrained loading till failure. As the deviatoric stress, q ($\sigma_1 - \sigma_3$) builds up with the strain (ϵ) a positive

pore water pressure is also generated (Fig. 4-1). Both the deviatoric stress, q and the pore water pressure, U simultaneously attain their maximum at the same strain, (point a in Fig. 4-1). When the sample is unloaded the pore pressure and strain reduce. However, even when the deviatoric stress q, reduces to zero, a positive residual pore water pressure persists corresponding to the unrecoverable deformation of the soil (point b in Fig. 4-1). This residual pore pressure (2.25 psi) is less than the peak pore pressure (2.95 psi) obtained at the end of loading cycle. When the sample is reloaded the same maximum deviatoric stress is attained accompanied by only a small additional strain. The stress strain curve for reloading rises vertically as compared to the previous virgin loading. The pore pressure also builds up during reloading, and attains a higher peak value (3.3 psi). Again after unloading (point d in Fig. 4-1) a residual pore pressure persists which is larger than obtained at the end of previous cycle. Also, an additional plastic strain is added to the sample. One more cycle produced a similar result. The peak pore pressure as well as residual pore pressure fall on straight lines which are more or less parallel and spaced apart.

Fig. 4-2 represents the stress path followed during the test. This is obtained by plotting successive states

of stresses in a stress space ($q = \sigma_1' - \sigma_3'$; $p = \sigma_1' + 2\sigma_3'/3$). The corresponding points are marked on this plot also. As it is apparent from the stress path, the peak strength obtained during each cycle stays constant and the stress path migrates towards left because of increase in pore pressure during cycling. Consequently,the subsequent failure points deviate from the original K_f-line.

Test TA-2:

In this test another identical sample, consolidated at 5 psi, was first subjected to 30 cycles of repeated stress at a level of 1.74 psi. Then, the sample was loaded close to failure and three cycles were applied at a stress level of 4.05 psi.

During cycling at the lower level of stress, there is slow but continuous increase in peak and residual pore pressure as well as in strain (Fig. 4-3). Stress-strain curves during cycling are very closely spaced and hence only loading and unloading points are marked for intermediate cycles. After about 25 cycles the increment in pore pressure per cycle begins to decrease. On stress path plot (Fig. 4-4) the gradual increase in pore pressure during cycling at this level leads to a slow migration of stress path towards K_f-line.

After cycling at smaller stress level, when the sample is reloaded, both stress-strain and stress-path rise vertically (Figs. 4-3 and 4-4). During cycling at close to failure stress level (4.05 psi), build up of pore pressure and development of strain are similar to those obtained in test TA-1. As is ap arent from the stress-path plot (Fig. 4-4), the second cycle itself builds up enough pore pressure to cause failure. The next cycle takes the stress path a little left to the K_f-line. Subsequent cycles further increase the pore pressure resulting in increased deviation in failure obliquity as the failure stress stays constant.

Test TA-3:

In this test of series A, fourteen cycles were applied at a deviatoric stress level of 2.56 psi, which is 53.2 % of the one-cycle failure stress.

During cycling at this level, both the peak and residual pore pressure as well as strain build up gradually. However, the increment in pore pressure and shown per cycle are observed to decrease with number of cycles. While the stress-strain curve seperates from the one cycle stress-strain curve and moves towards right, the pore pressure_ strain curve moves up and shifts to the left of one-cycle

pore pressure strain curve (Fig. 4-5).

The same sample was reloaded to a stress level of 3.75 psi and cycled at this level. During reloading both stress-strain curve and stress path (Figs. 4-5 and 4-6) rise almost vertically. Again, cycling at this higher stress level, subsequent increase in pore pressure moves the stress path to the left and takes it close to the K_f-line. As it is obvious from the Figure 4-6, only one or two more cycles would have generated enough additional pore pressure to induce failure.

All other behavioural aspects are more or less similar to those described in Tests TA-1 and TA-2.

4.3 Series B: Normally Conslidated at σ_c' = 30 psi.

Test TB-1:

In this test, a sample normally consolidated at 30 psi was subjected to three cycles each leading to failure. The corresponding loading and unloading points (a , b , c ..etc. are marked on stress-strain curve, pore pressure-strain curve and stress path (Figs. 4-7 and 4-8).

The results obtained from this test are essentially similar to the cycled-to-failure test of series A (TA-1). But in this test, the subsequent failure stresses are observed

to decrease. The increase in pore pressures moves the stress path to the left as usual but the subsequent failure points are observed to follow the same failure obliquity (Fig. 4-8).

Test TB-2:

Another sample of this series was subjected to twenty cycles at a stress level of 4.42 psi ie. 23 % of the one cycle failure stress.

The behavioural feactures observed are very much similar to these observed during test TA-2. Only the magnitudes involved are different in the two tests. Here also, a reduction in increment of pore pressure and strain per cycle are observed.

When the sample is reloaded after cycling at this level, stress path (Fig. 4-10) rises vertically. Although, the sample could not be loaded to failure a probable extension of the stress path reveals that the sample would fail at the same one cycle maximum stress difference. Also, the stress path will terminate at the same point on K_f-line.

Test TB-3:

A repetitive stress of 9.00 psi, was applied to the sample. Fourteen cycles were applied at this level

and the sample was then loaded to failure (Fig. 4-11).

Pre-failure cycling produces almost similar results as obtained in the test TA-3. Then the sample is loaded to failure after pre-failure cycling, the stress path rises vertically in begining and then merges with the single-cycle loading stress path and follows the same till failure is reached (Fig. 4-12).

Test TB-4:

In the last test of this series, the sample was subjected to a stress difference of 14.2 psi ie. 74 % of one cycle failure stress.

The increments in pore-pressure and strain per cycle reduce very much in the fifth cycle itself. In few more cycles, the peak pore pressure and strain would have attained their maximum limiting values. The sample could not be loaded to failure after pre-failure cycling. However the extension of vertically rising stress path (Fig. 4-14) shows that it will shoot up almost to the one cycle failure point C on K_f-line.

4.4 Series C: Normally consolidated at $\sigma_c' = 45$ psi.

Test TC-1:

The sample consolidated at an effective confining pressure of 45 psi was cycled to failure. The results are

exactly similar to those of test TB-1. Here also, a decrease in subsequent failure stress is observed and the failure obliquity is found to remain unchanged (Figs. 4-15 and 4-16).

Test TC-2:

In this test the cycled stress level was 7.24 psi ie. 25.5 % of the one cycle failure stress (q_f= 28.3 psi). Pre-failure cycling response is very much similar to that observed in test TB-2. When the sample is subsequently loaded to failure stress path rises vertically, meets the one_cycle stress path before failure and then follows the same till failure is achieved at the same maximum stress difference and stress obliquity (Fig. 4-18).

Test TC-3:

This is the last test of the series C. The sample was subjected to a cycle stress level of 14.72 psi. The pre-failure cycling behaviour is similar to that observed in all other tests. Only five cycles could be applied because of time limitation. The reduction in per cycle pore pressure and strain increments is rapid. Post-cycling behaviour is similar to that observed in test TB-3 (Figs. 4-19 and 4-20).

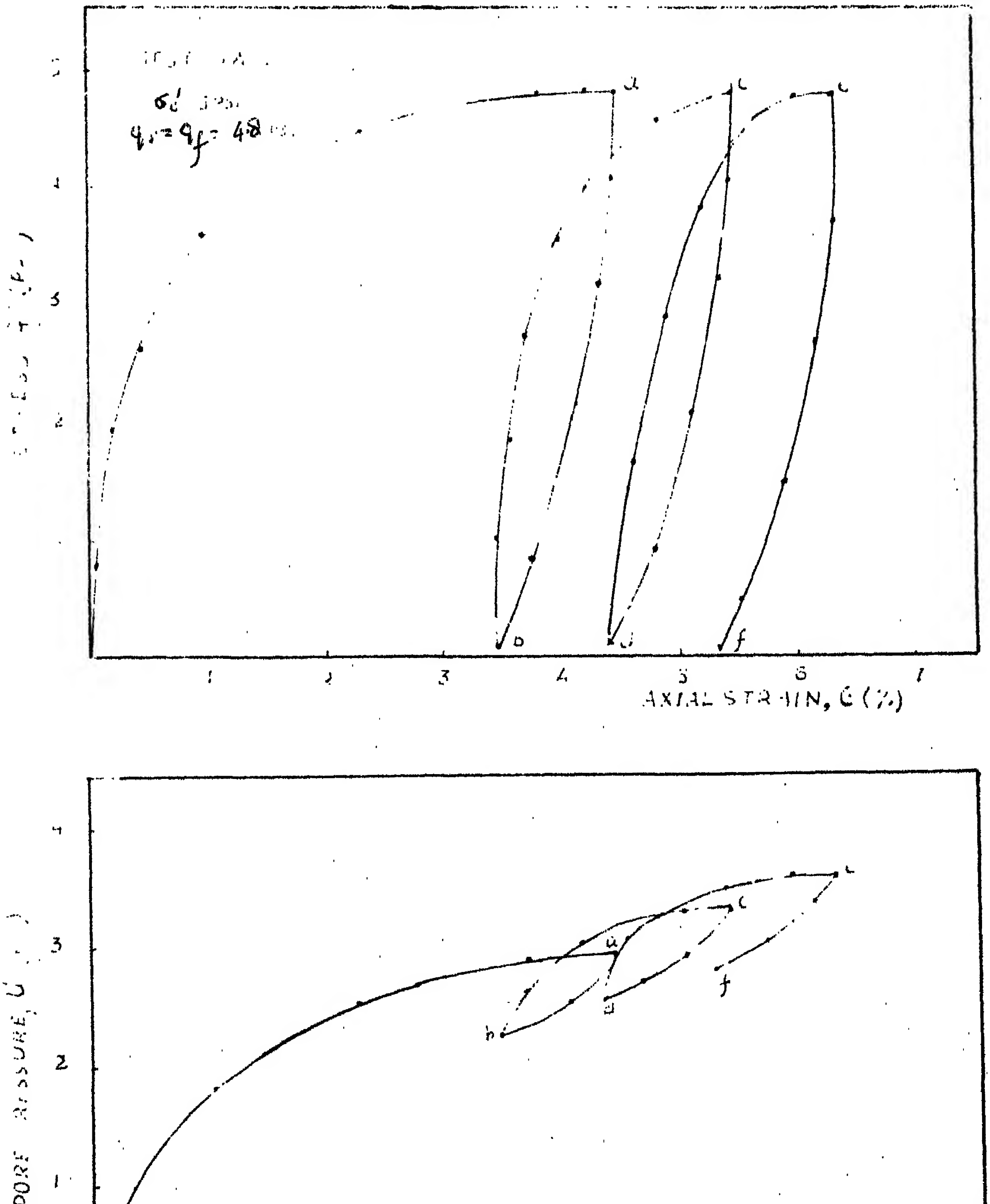


Fig. 4-1. Stress-Strain & Pore Pressure-Strain Curves For Cycled At Failure Test, TA-1

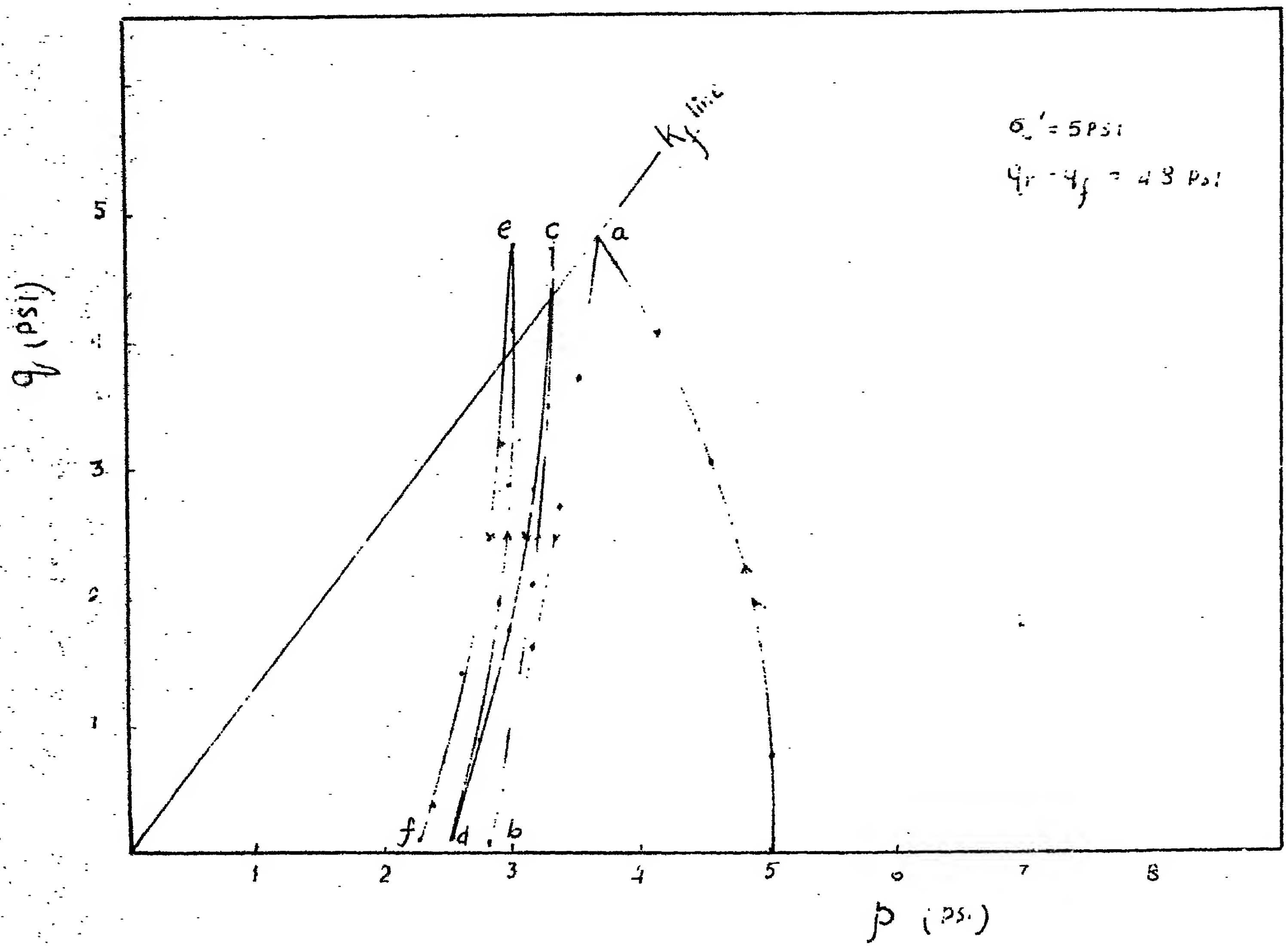
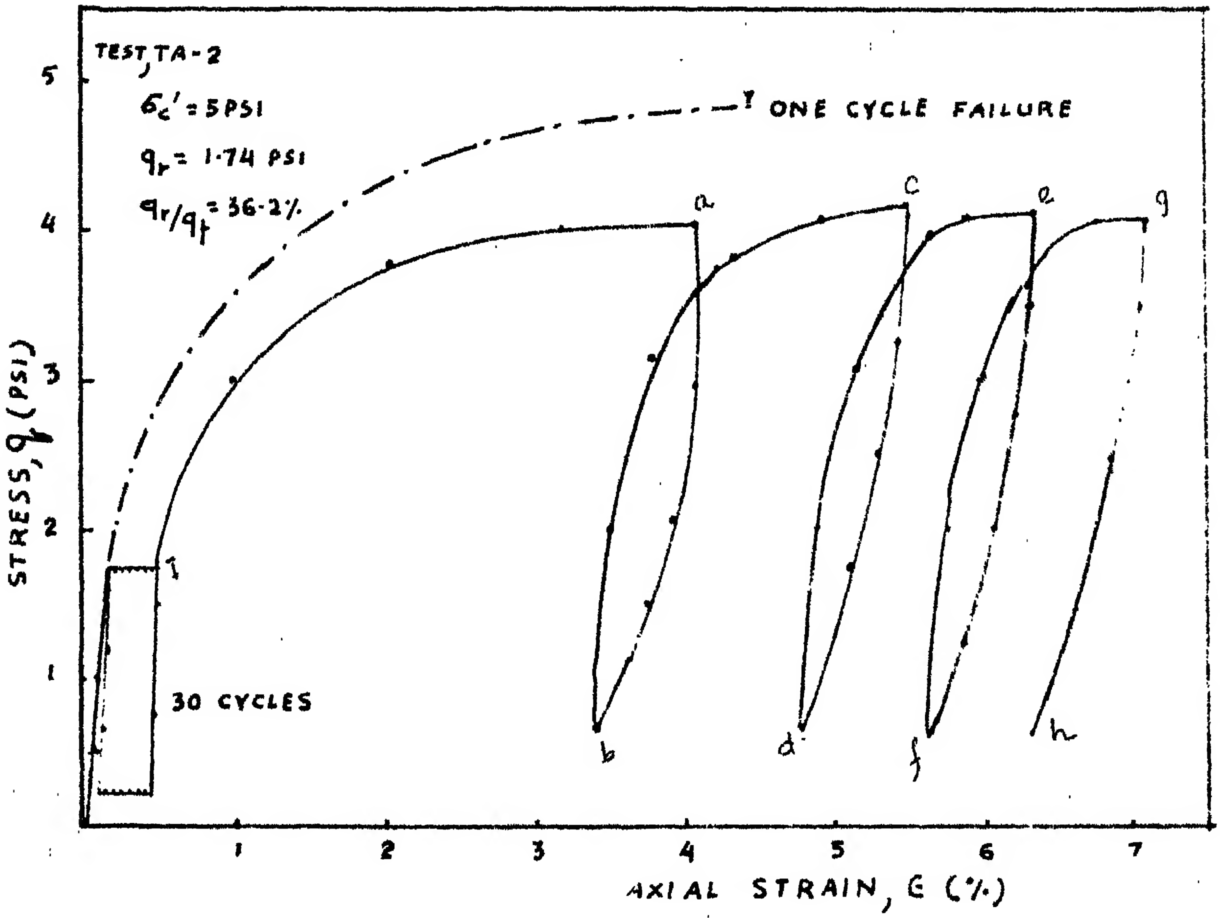


Fig. 4-2. Stress Path For Cycled At Failure Test, TA-1



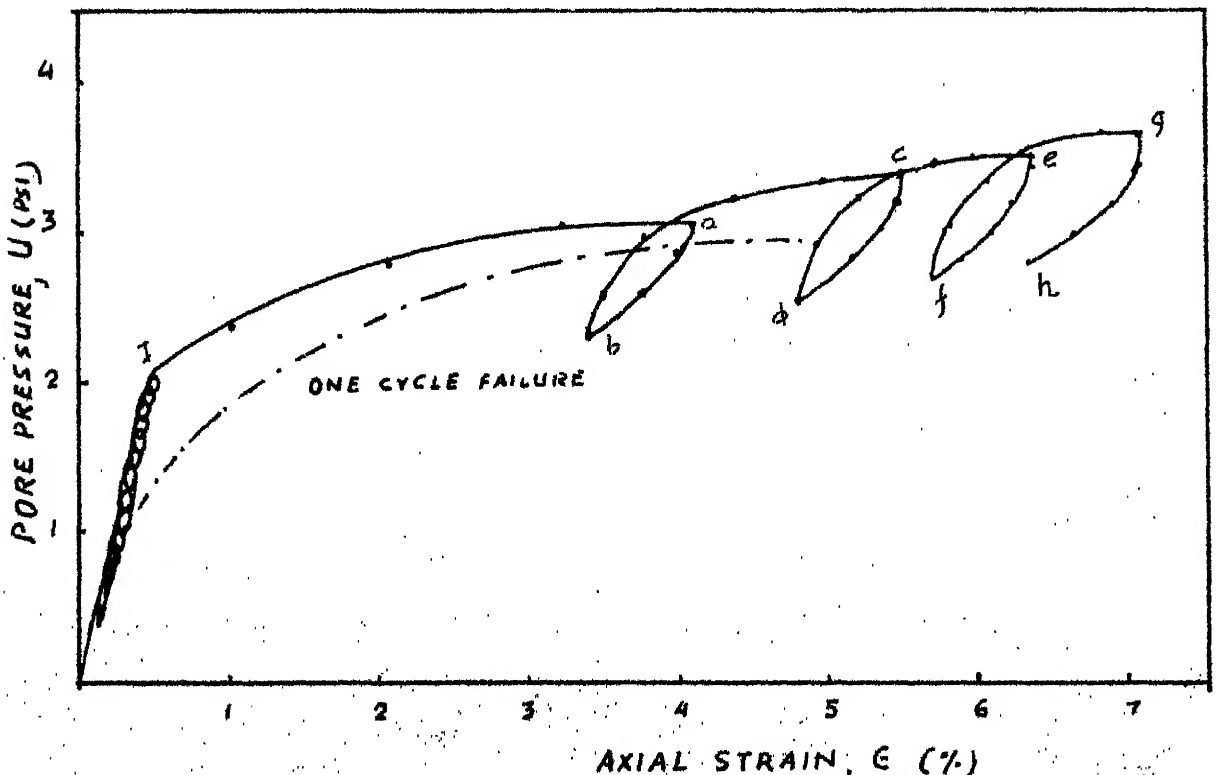


Fig. 4-3. Stress-Strain & Pore Pressure-Strain Curves For The Test, TA-2.

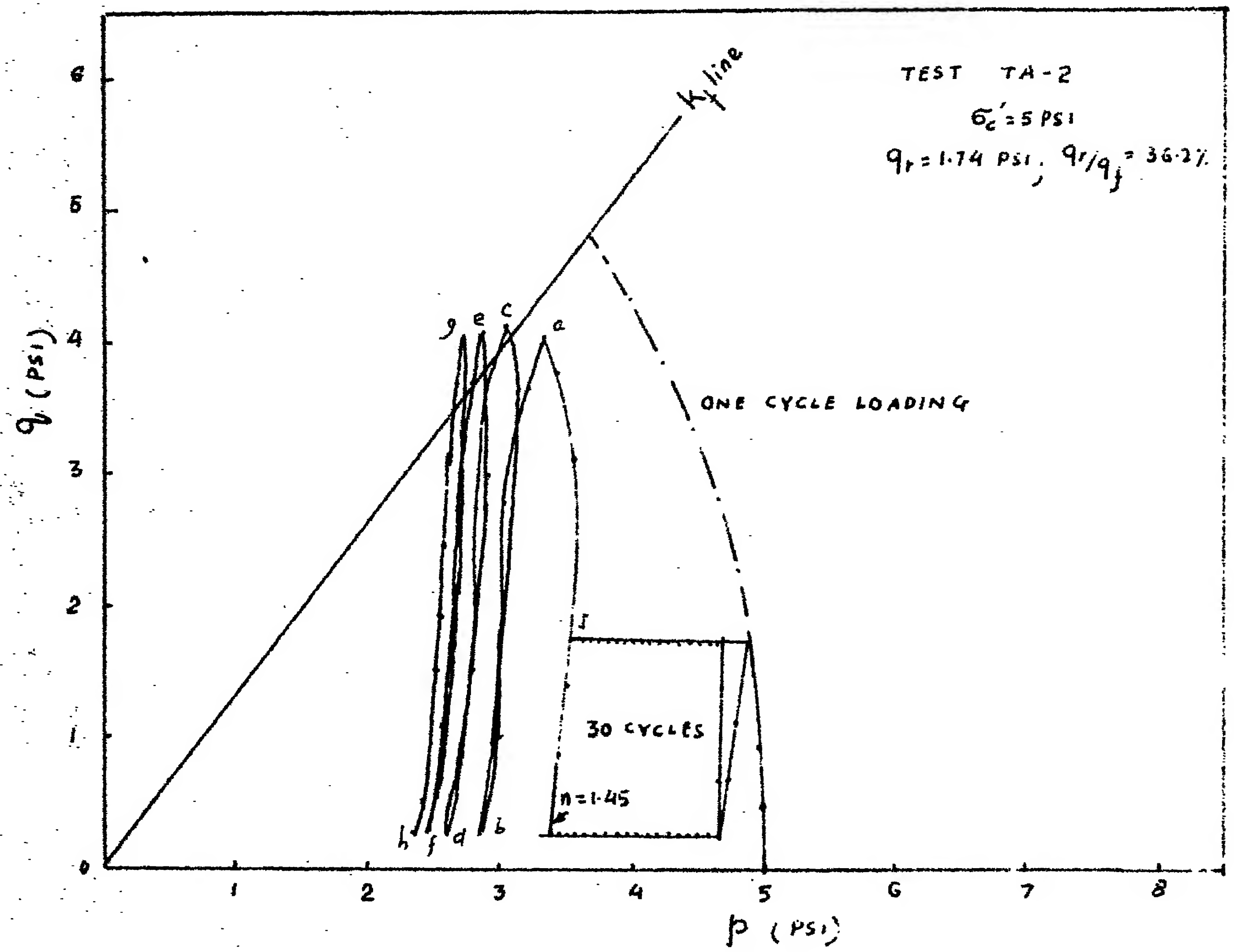


Fig. 4-4; Stress Path For The Test, TA-2

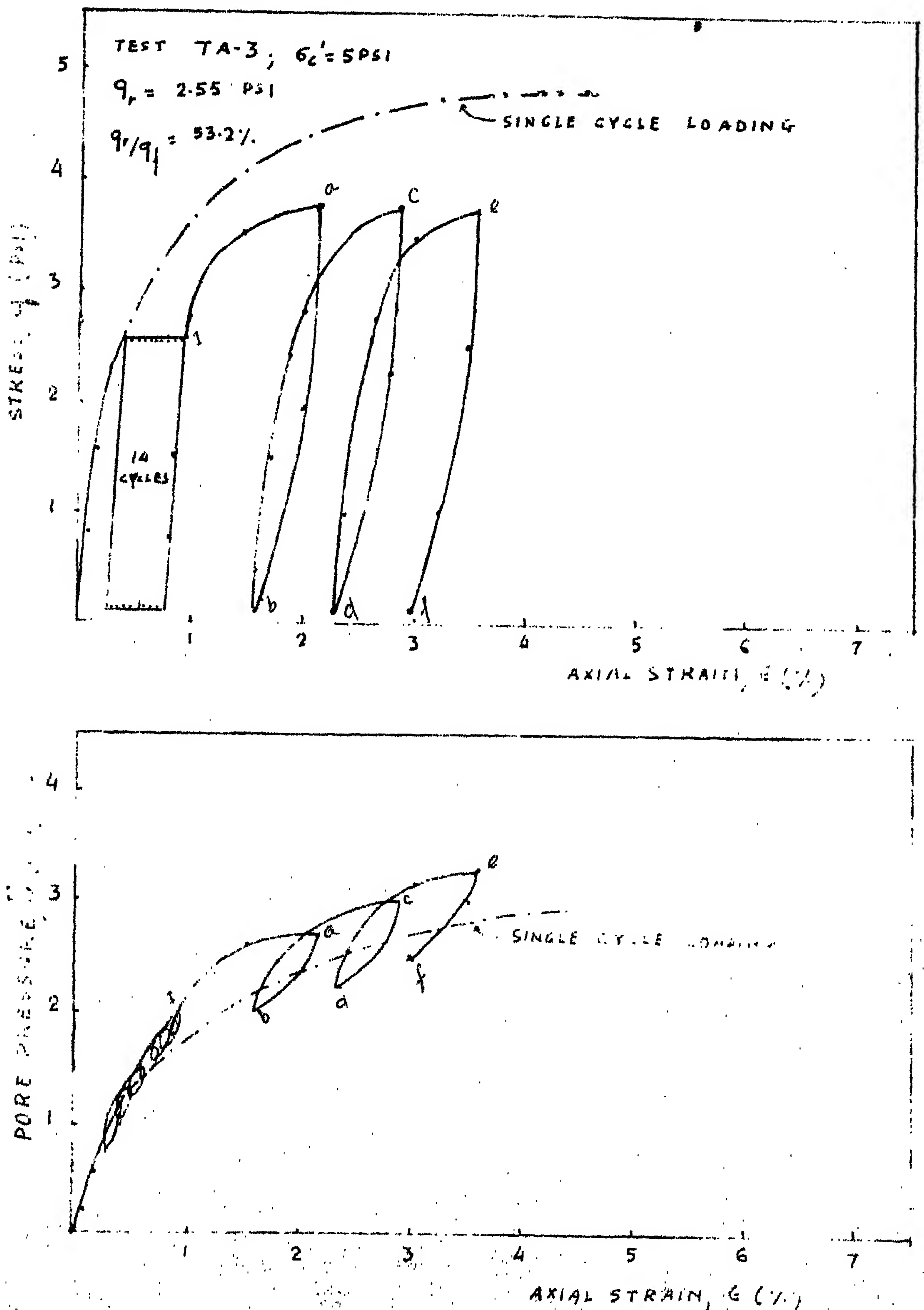


Fig. 4-5 Stress-Strain & Pore Pressure-Strain Curves For The Test, TA-3.

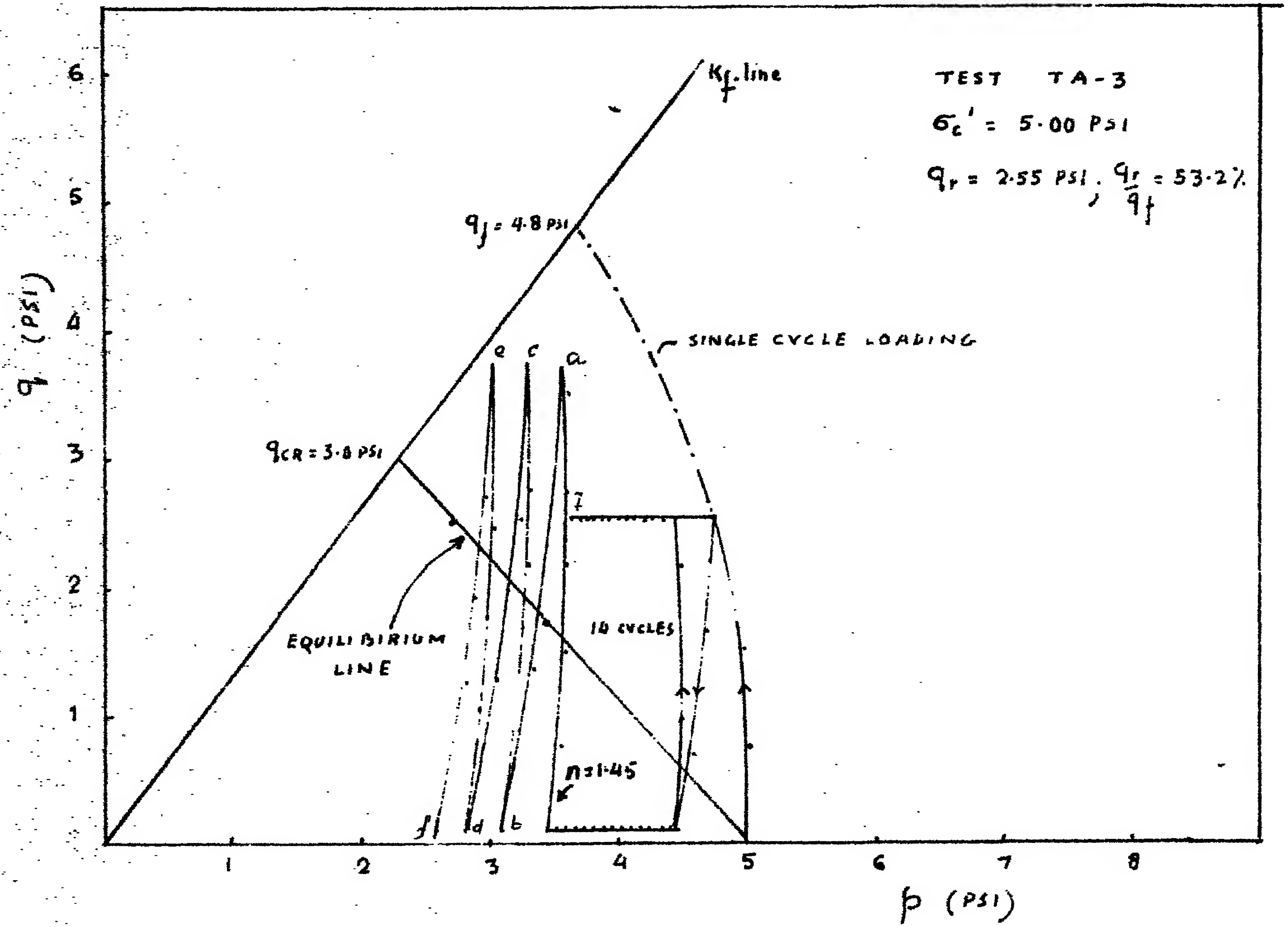


Fig. 4-6: Stress Path For The Test, TA-3.

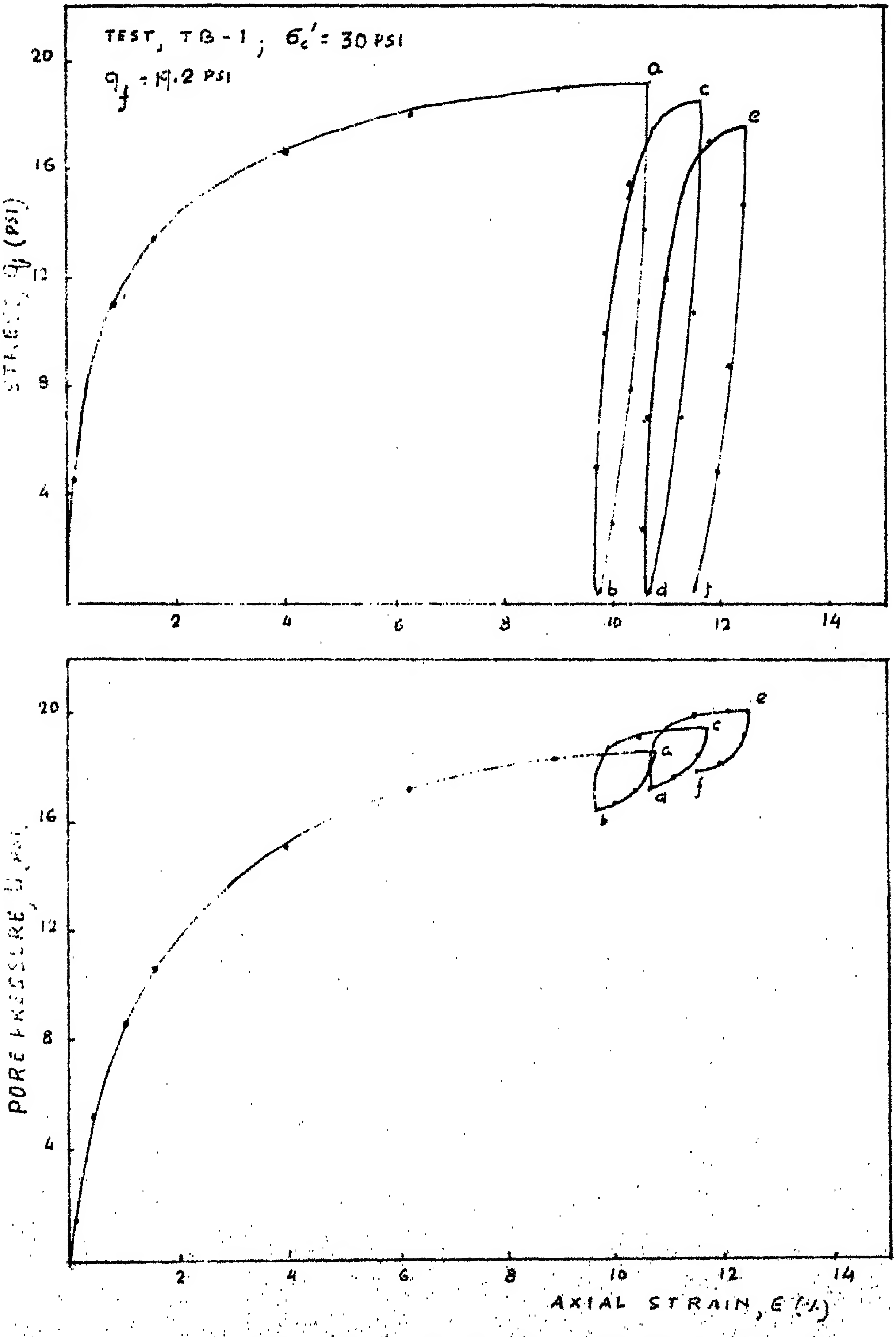


Fig 4-7. Stress-Strain & Pore Pressure-Strain Curves

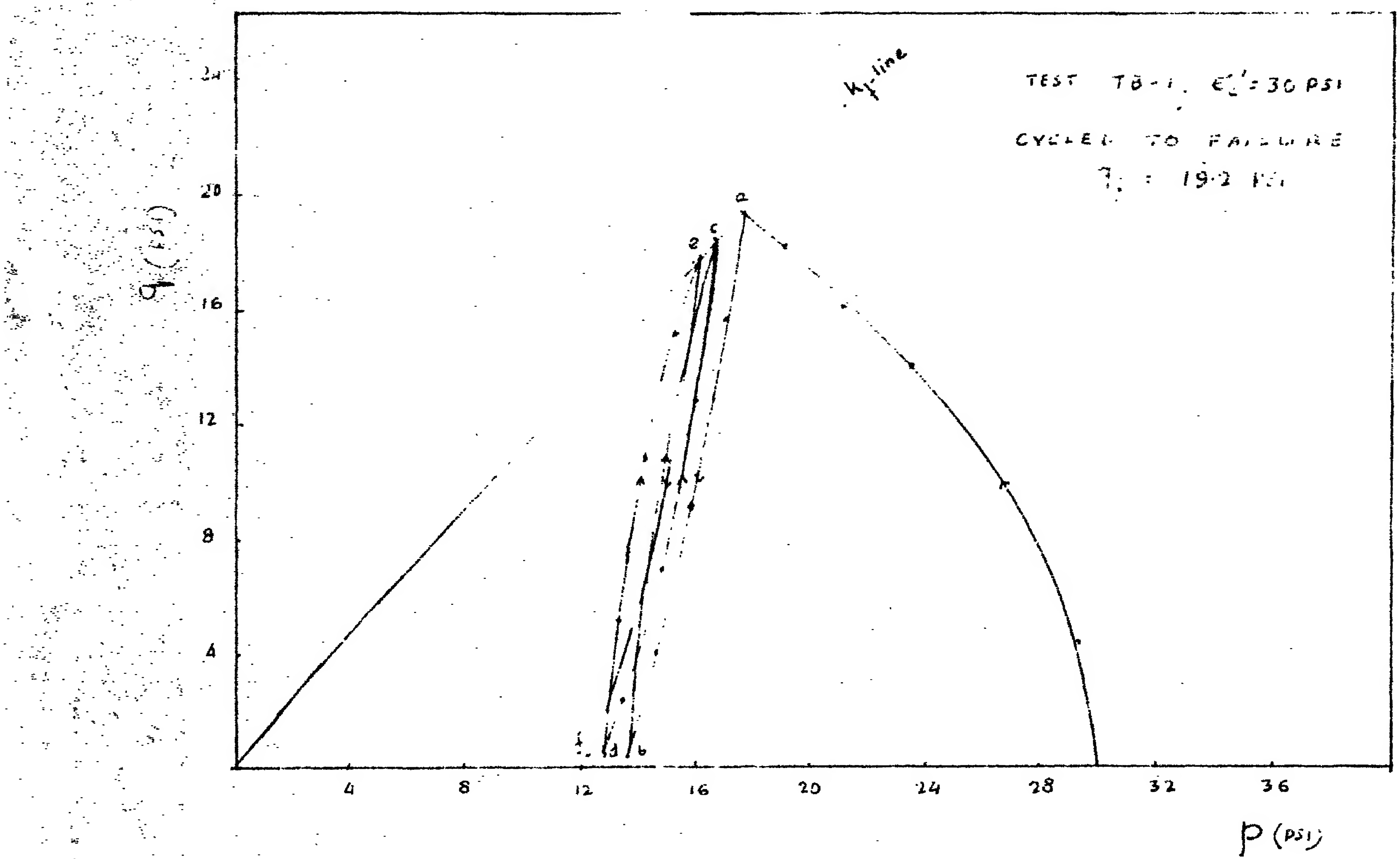


Fig 4-8. Stress Path For Cycled-At Failure Test, TB-1

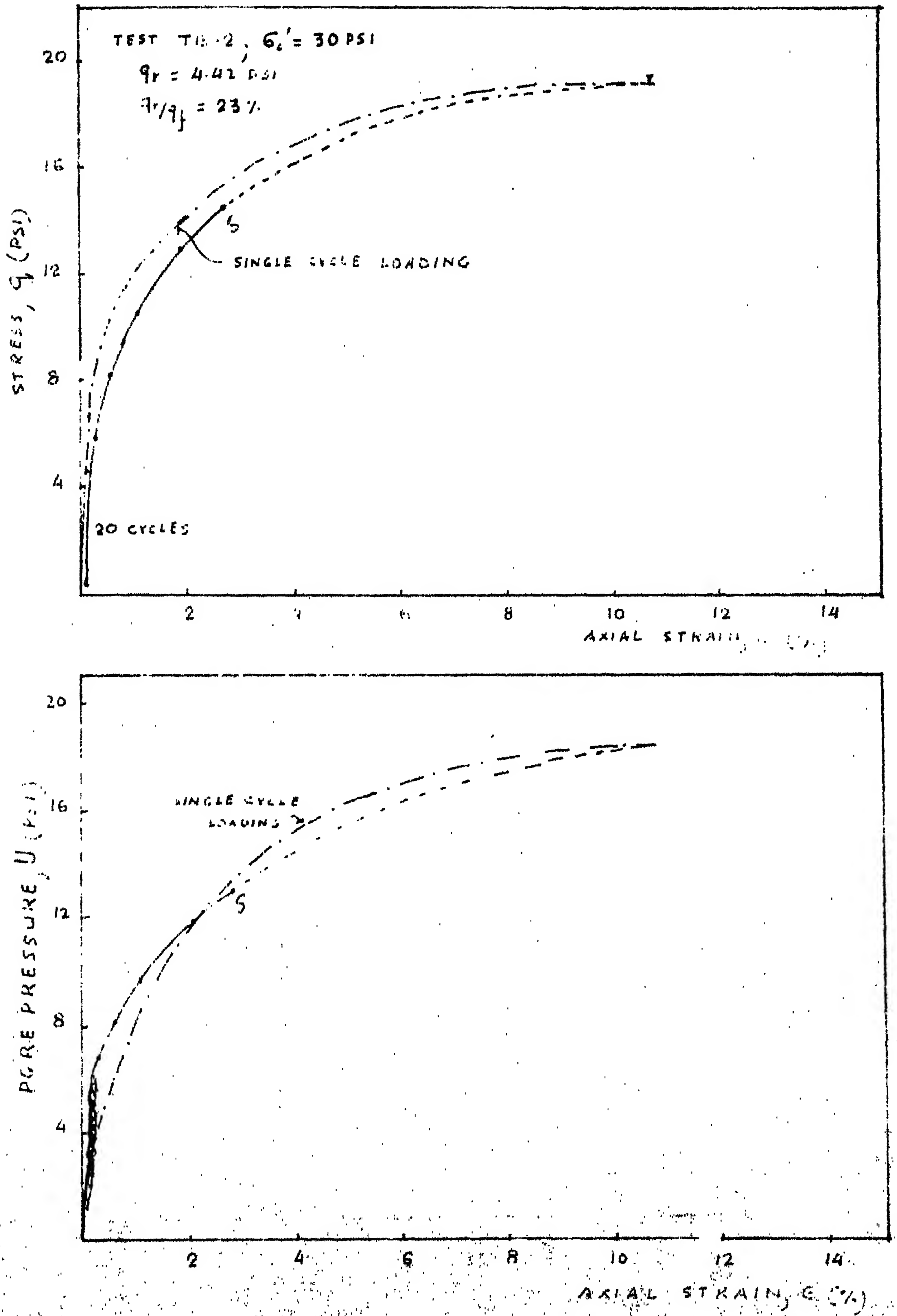


Fig 4-9. Stress-Strain & Pore Pressure - Strain Curves For The Test, TB-2.

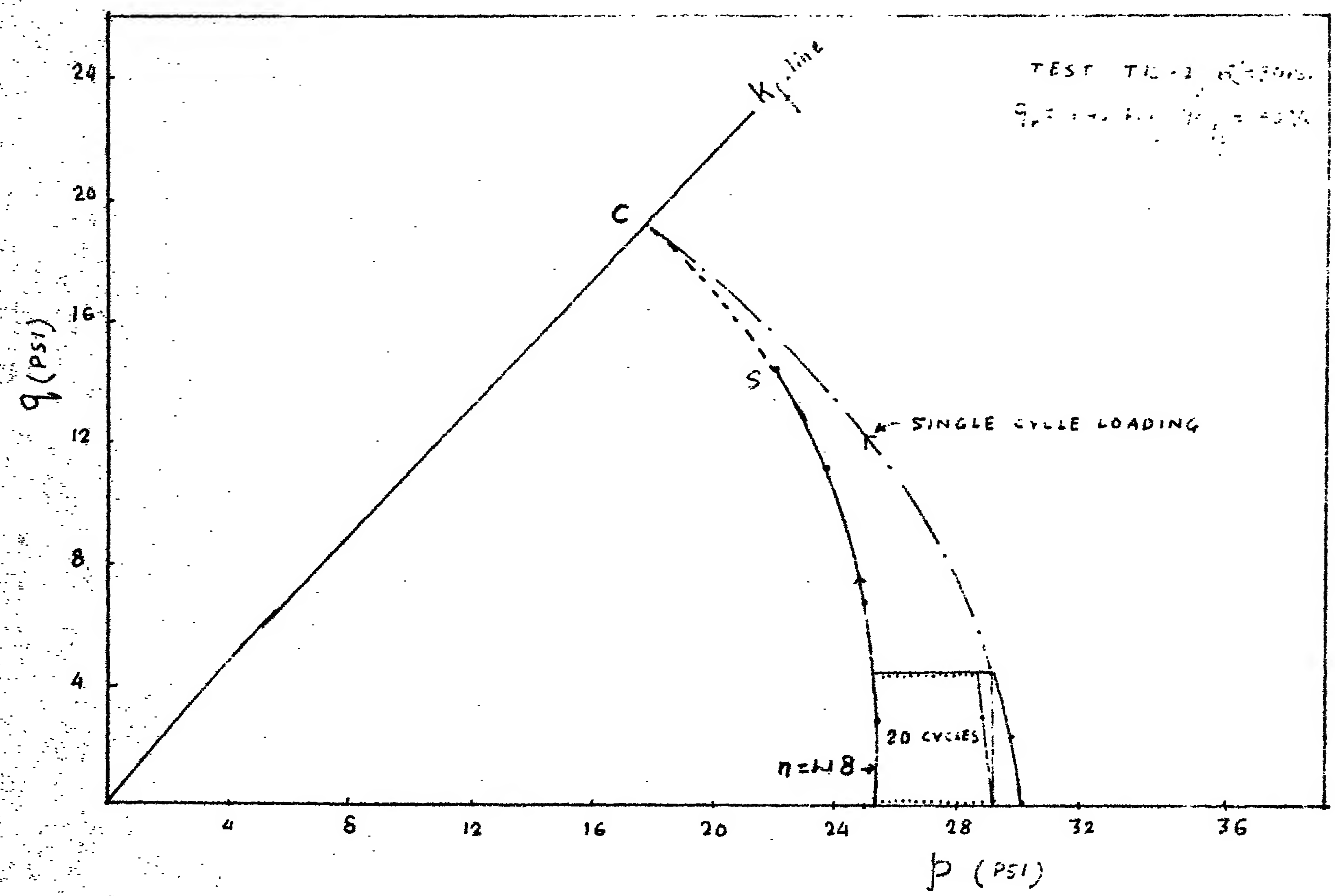


Fig. 4-10. Stress Path For The Test, TB-2.

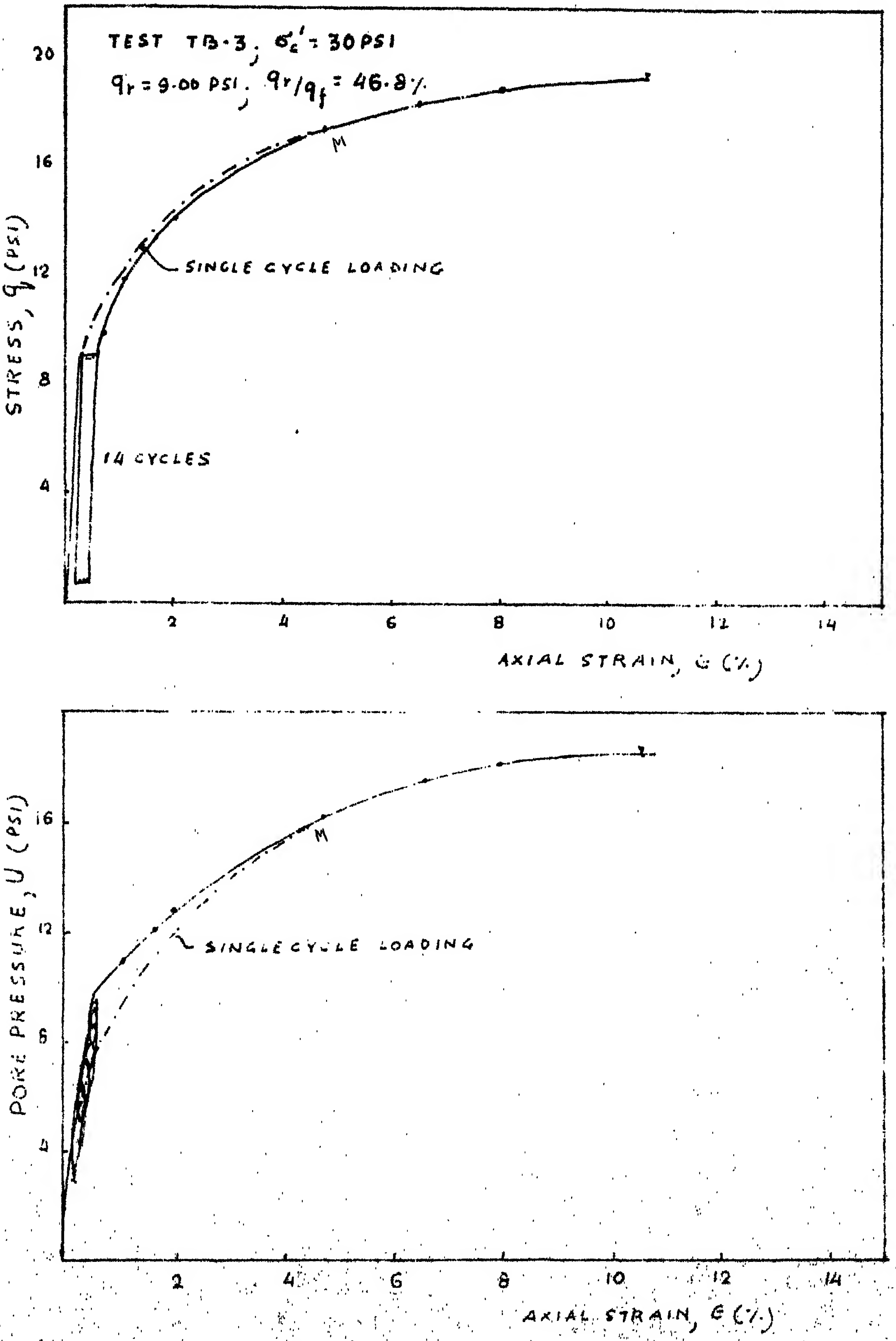


Fig. 4-11. Stress-Strain & Pore Pressure-Strain Curves For The Test, TB-3

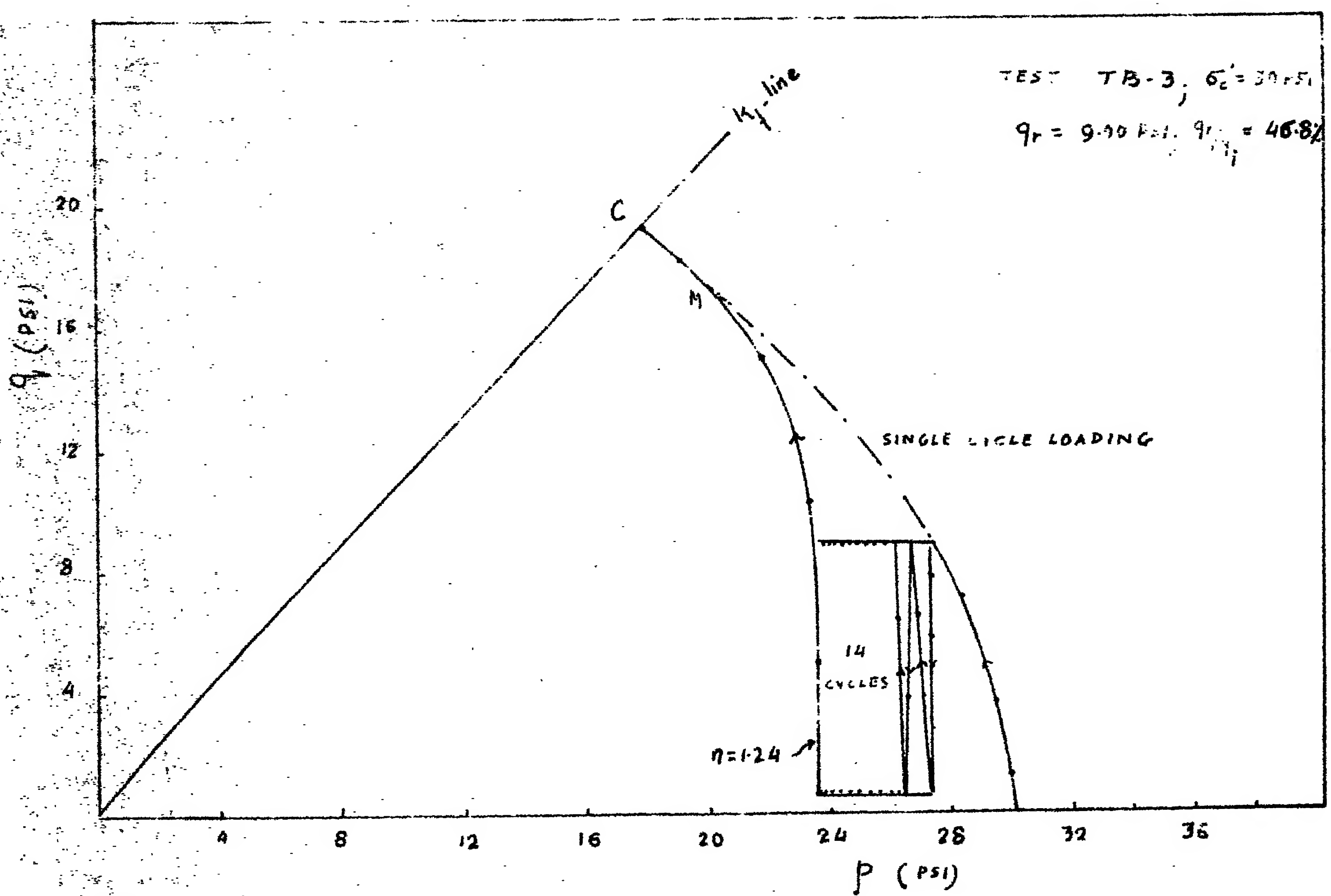


Fig. 4-12. Stress Path For The Test, TB-3.

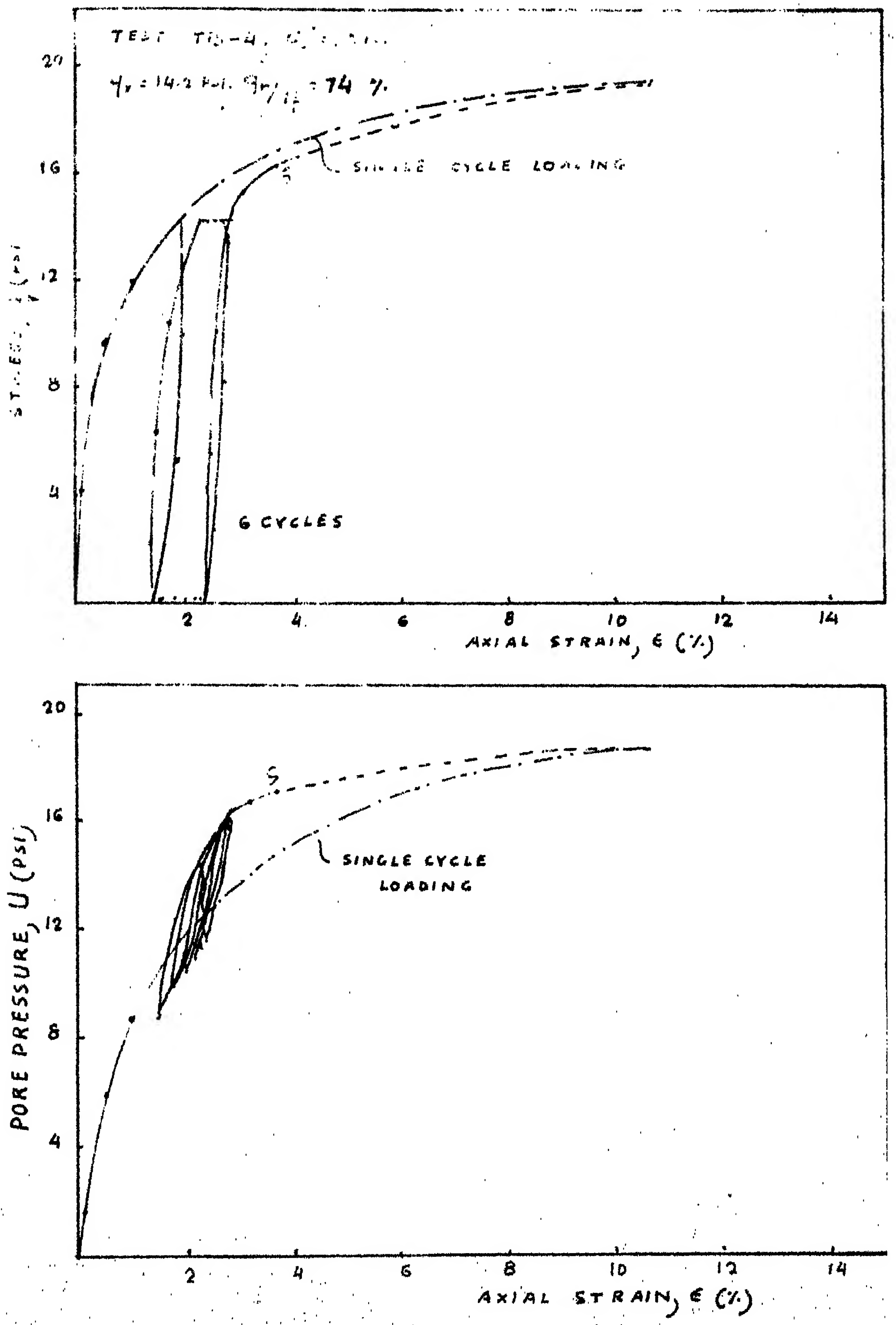


Fig. 4-13. Stress - Strain & Pore Pressure - Strain Curves For The Test, TB-4.

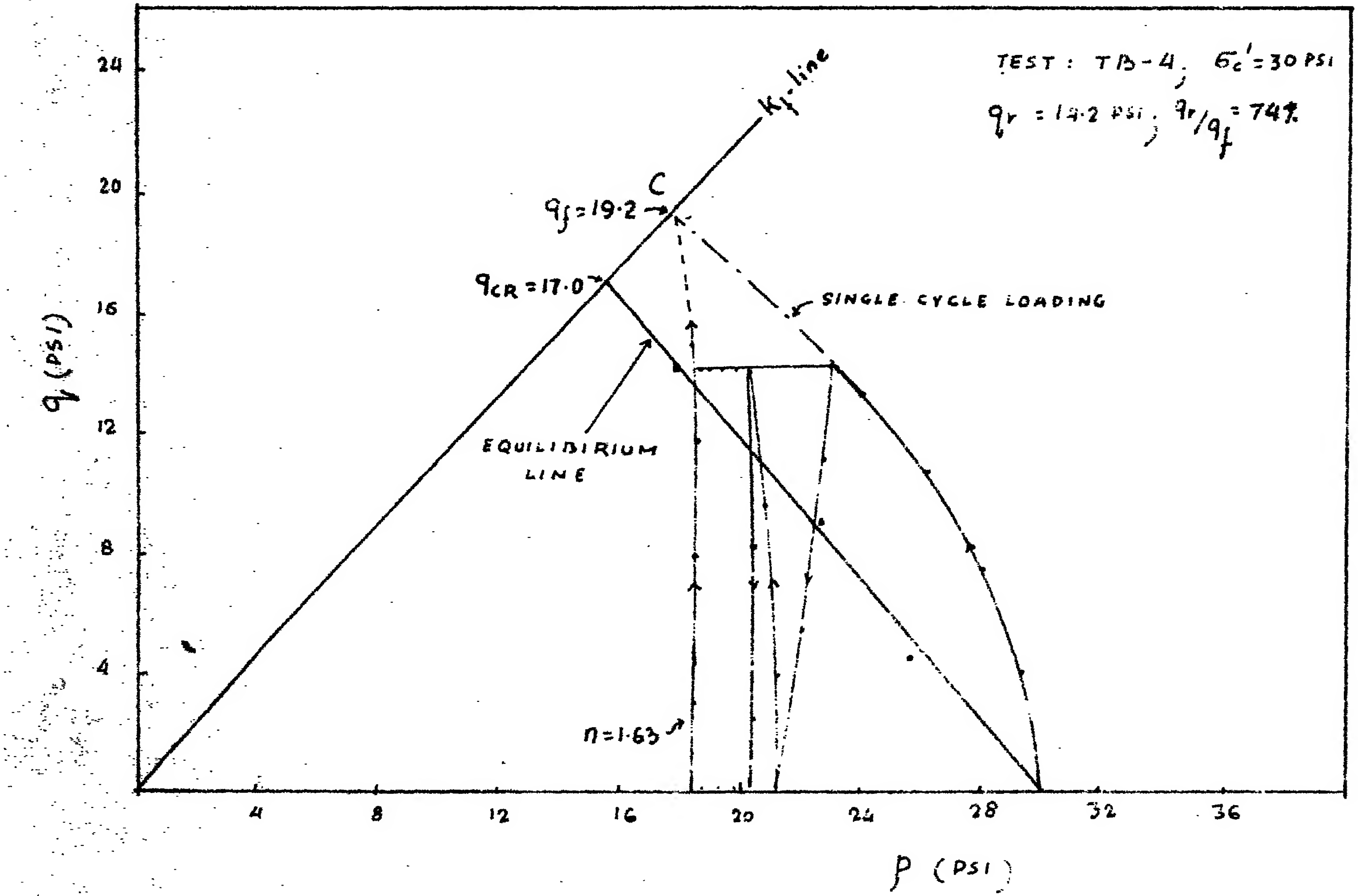


Fig. 4-14. Stress Path For The Test, TB-4.

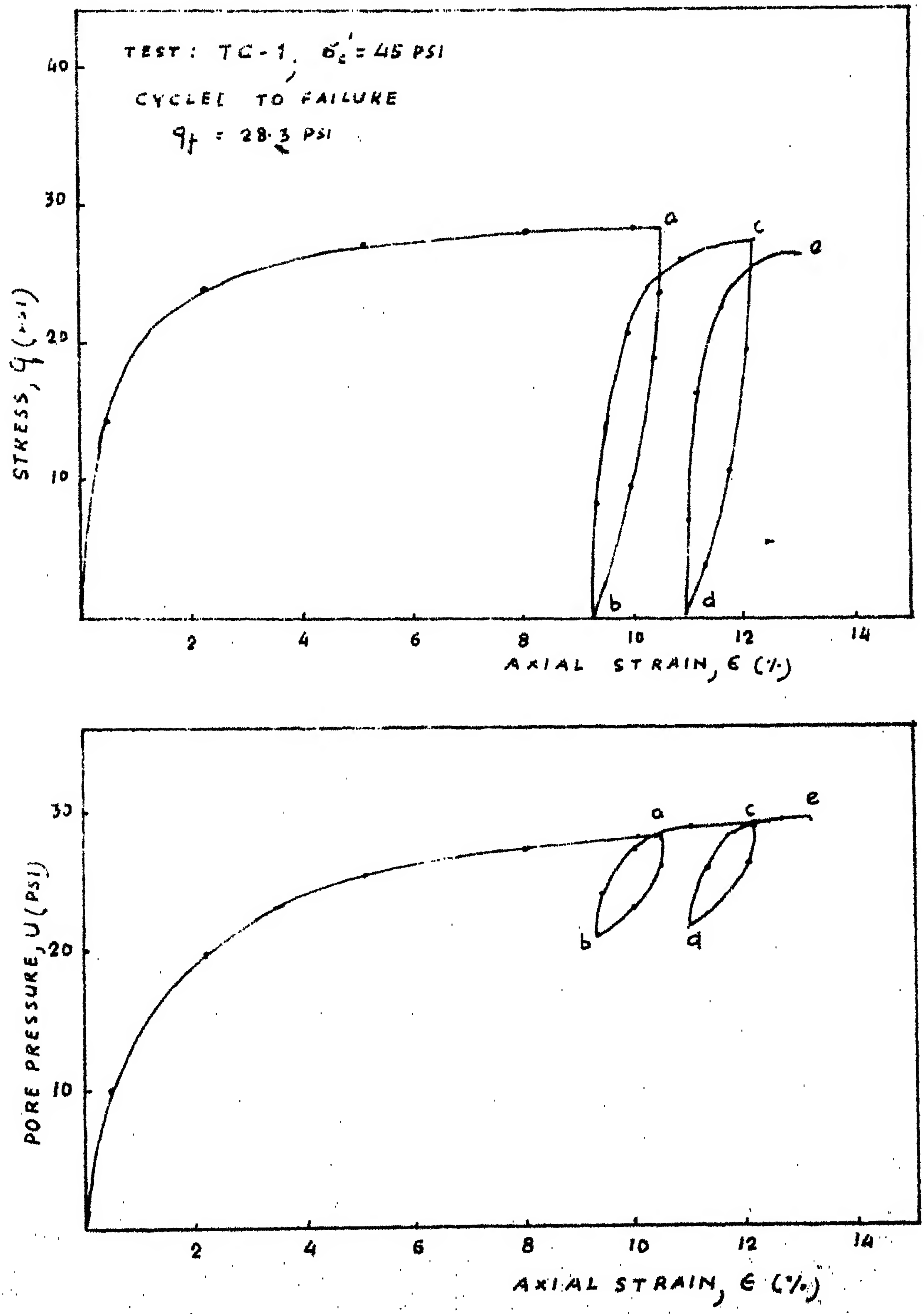


Fig. 4-15. Stress-Strain & Pore Pressure Strain Curves For Cycled-At-Failue Test, TC-1.

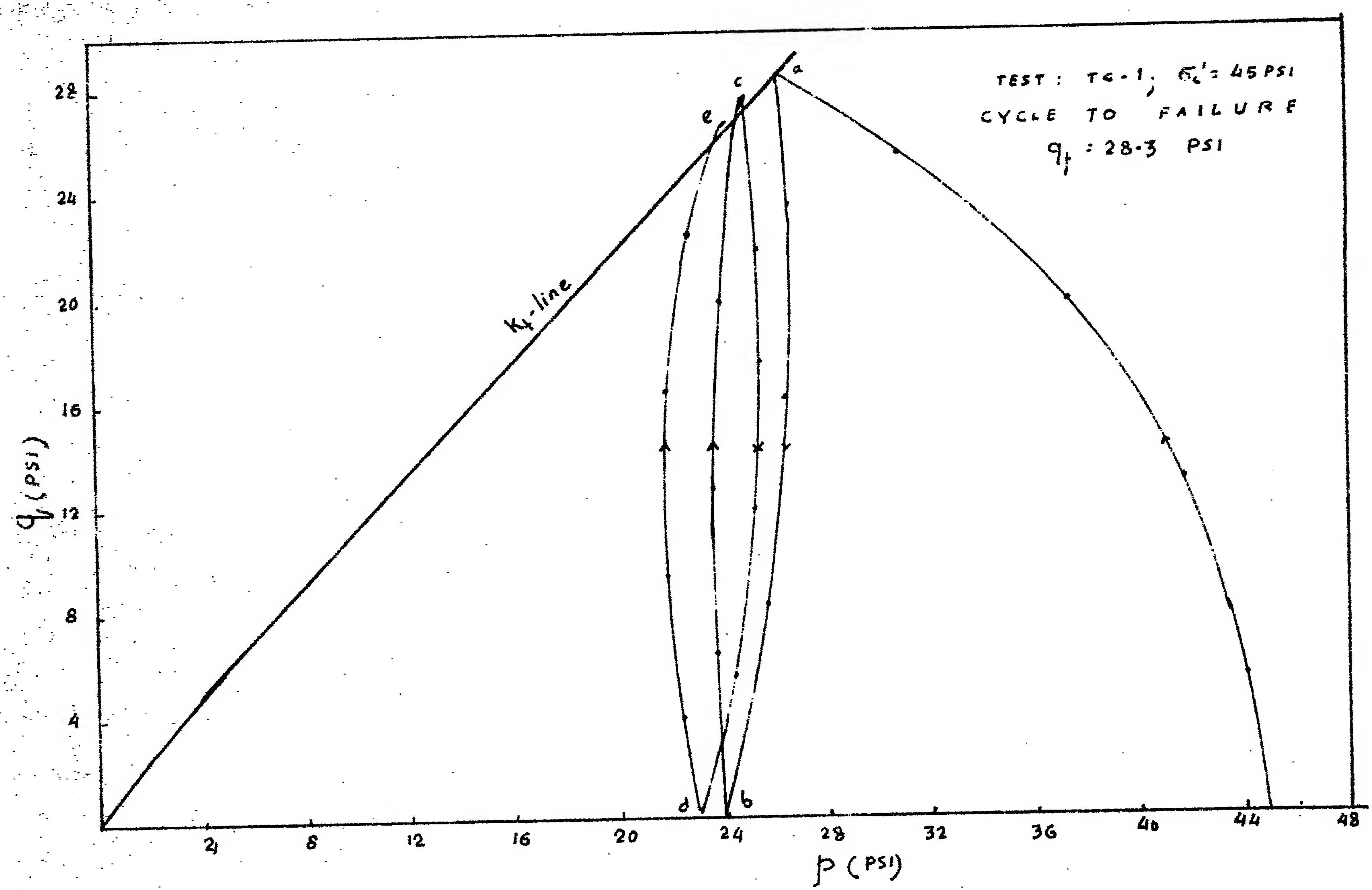


Fig. 4-16. Stress Path For Cycled-At-Failure Test, TC-1.

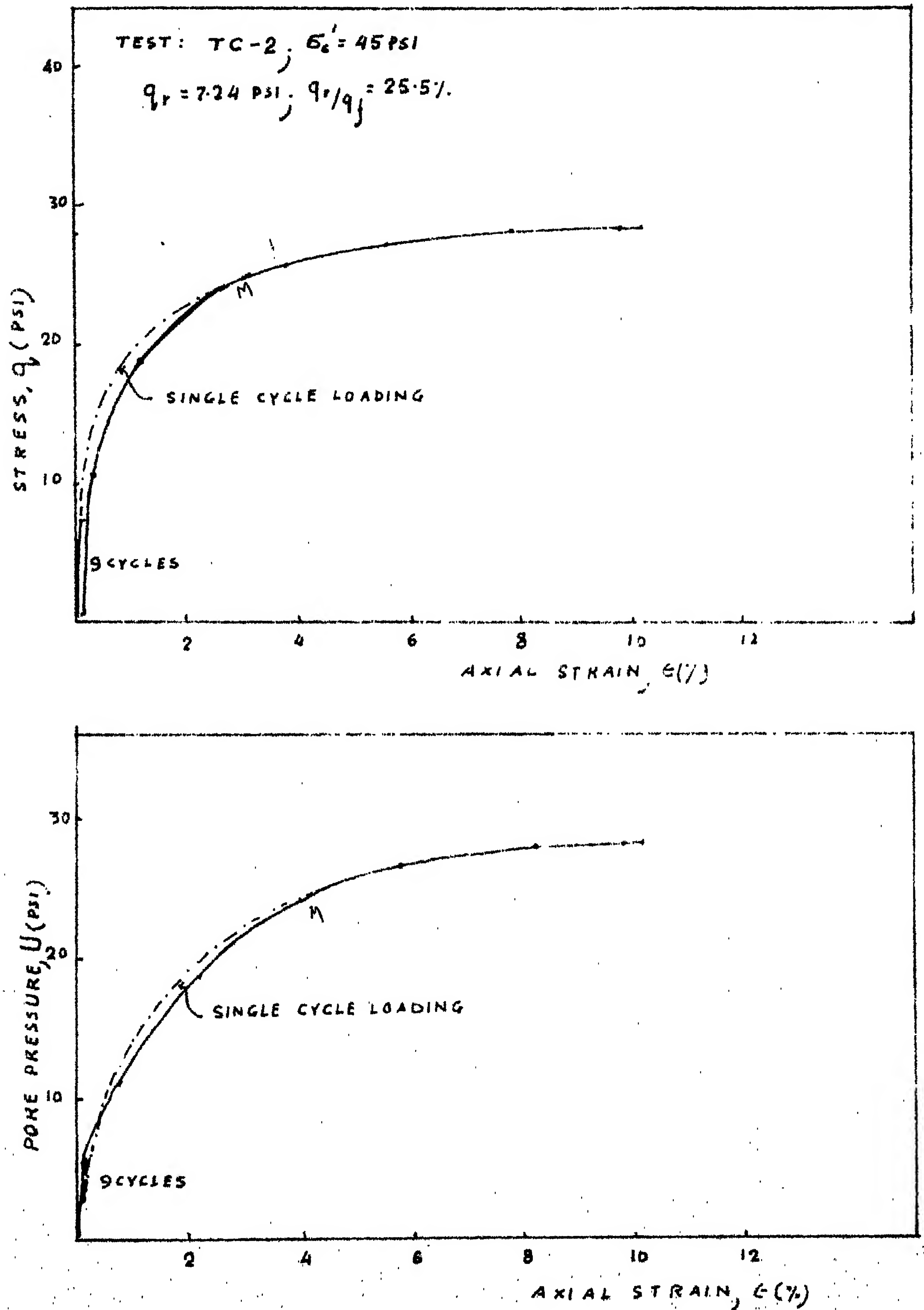


Fig. 4-17. Stress-Strain Pore Pressure - Strain Curves For The Test TC-2.

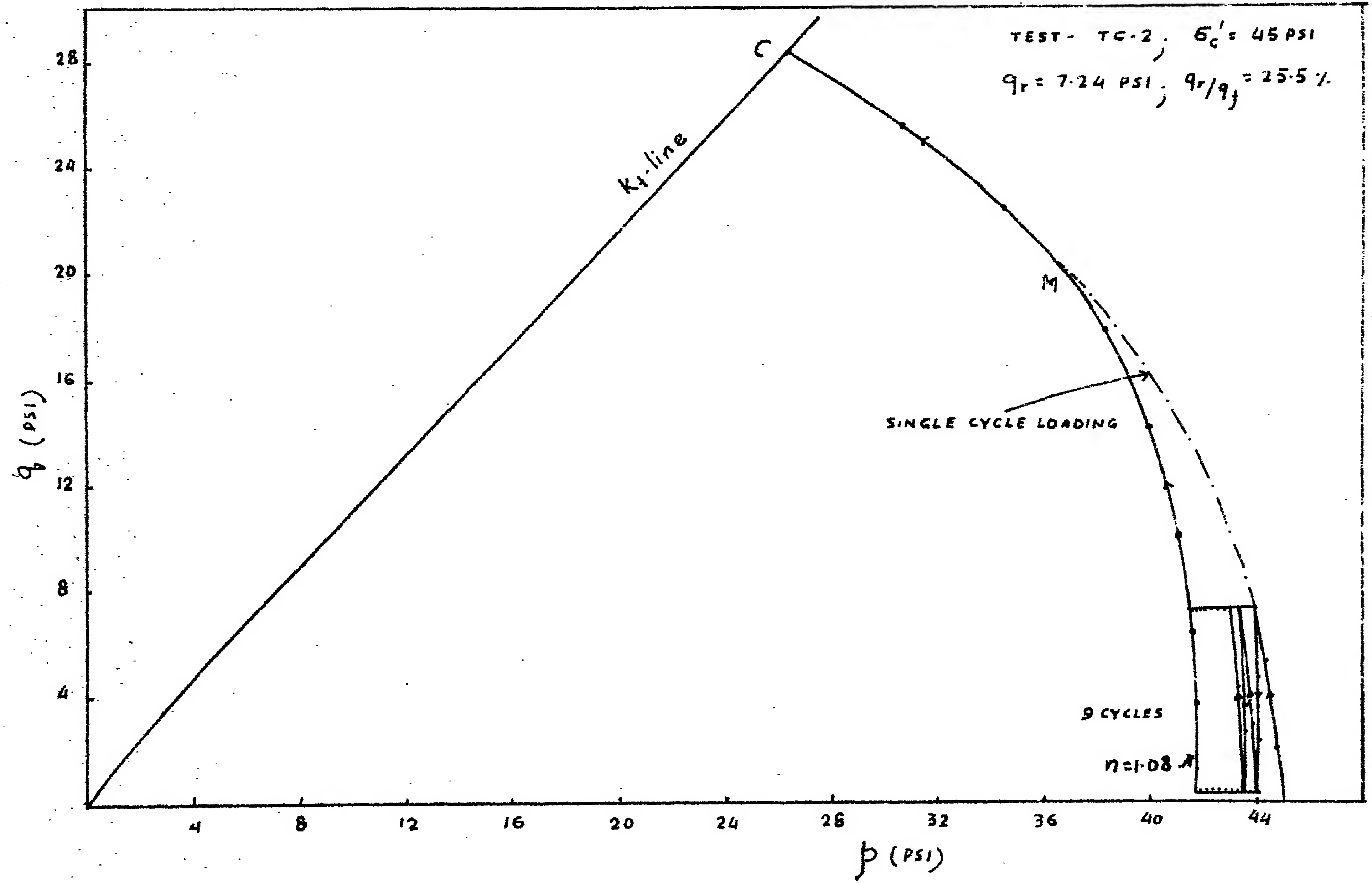


Fig. 4-18. Stress Path For The Test, TC-2.

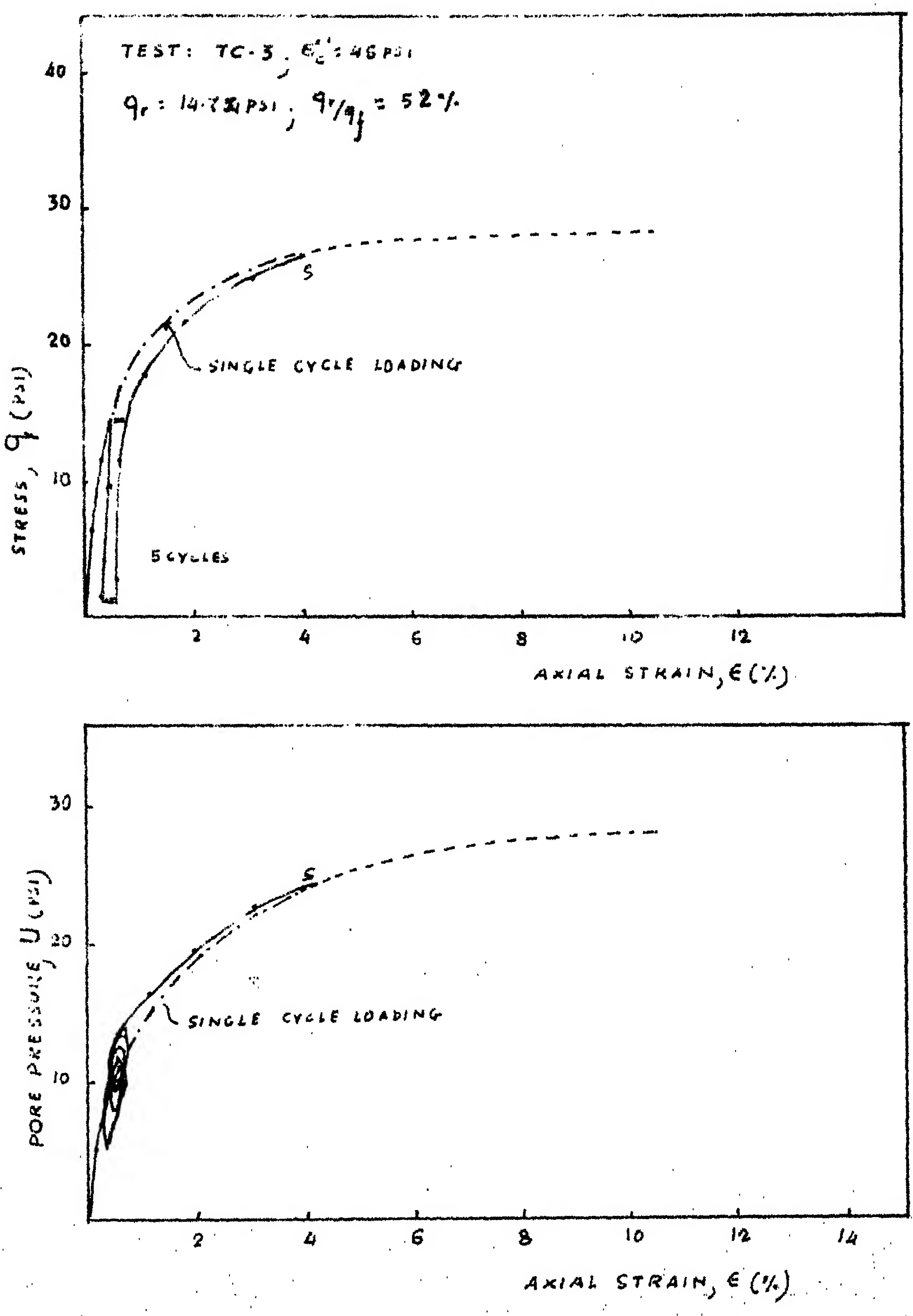


Fig. 4-19. Stress-Strain & Pore Pressure - Strain Curves For The Test TC-3.

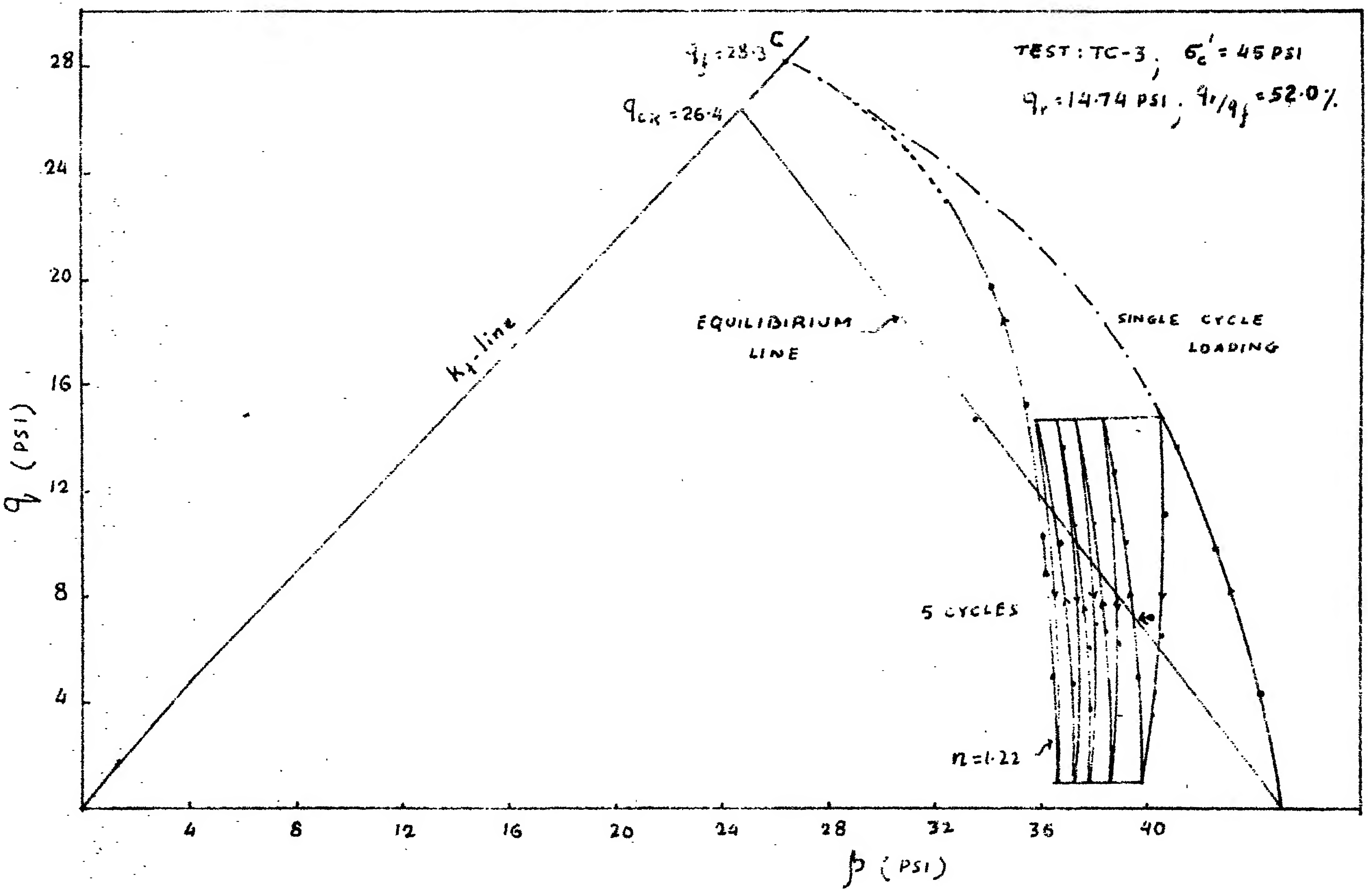


Fig 4-20. Stress Path For The Test, TC-3.

CHAPTER V

DISCUSSION OF TEST RESULTS

5.1 Introduction:

In chapter V the results of test series were presented. Here, in this chapter all the behavioural features of the soil subjected to repeated loading are discussed and analysed. Also, each particular behavioural pattern is explained by reasoning based on mechanistic picture of soil structure - pore fluid interaction.

The pore pressure and strain response during cyclic application of stress, the critical level of cyclic stress for repeated loading and post cycling behaviour are discussed separately.

5.2 Pore Pressure and Strain Response During Cyclic Applicatic of Stress:

General Observations:

When the saturated normally consolidated clay is subjected to a deviatoric stress increment both strain and a positive pore pressure build up simulataneously. During unloading only a small portion of the total strain is recovered and therefore at the end of unloading a positive residual pore pressure persists. On subsequent reloading to the original stress level, the soil undergoes an additional deformation

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and so also an additional pore pressure is induced. Again, on unloading a larger residual pore pressure persists corresponding to the increased amount of unrecoverable strain. Similar response of pore pressure and strain is obtained on subsequent cycling at the same deviatoric stress. For the same cycle, the residual pore pressure and strain at the end of unloading are always found to be less than the peak pore pressure and strain achieved at the end of loading. Moreover the pore pressure - strain curves are different for loading and unloading ie. the relationship between pore water pressure and strain is not unique but dependent on direction of stress path.

This particular behavioural feature is observed to be general in nature irrespective of cycled stress level and confining pressure (Figs. 4-1 to 4-20).

Lo (1961) observed a unique relationship between strain and pore pressure in a saturated clay subjected to repeated loads. His data can however be interpreted in another way. The pore pressure versus strain plots for all cycles after the first may in fact be open loops, the unloading portion of the curve not retracing the previous loading cycle. The single curve which Lo has drawn is perhaps the average of many of these pore pressure - strain loops.

Sangrey (1968), on the contrary reported a build up of pore pressure during cycling resulting in much higher value than the one obtained during single - cycle loading. This observation is similar to that noted in the present study. However, he observed that the peak of pore pressure was achiev somewhere before the end of loading and that, residual pore pressure at the end of unloading was higher than the pore pressure obtained at the end of previous loading cycle. He took it as a characteristic feature of his soil without givin any explaination for it. His samples were undisturbed, therefore this peculiar response might have resulted from a special structure of the soil built in it by overconsolidation in the field.

It is believed that the pore pressure-strain response during cyclic application of stress, as observed in the study is representative of saturated normally consolidated clays.

Mechanistic Picture:

The development of pore water pressure during undrained loading is a consequence of large disparity between the compressibilities of soil skeleton and pore water. The soil skeleton is highly compressible as compared to pore water. The compressibility of soil skeleton is controlled by interaction between individual soil particles, specially by inter particle sliding. Depending upon the stress level, the

deformation resulting from bending of plate like clay particles is wholly or partly recoverable, whereas the strain contributed by interparticle sliding is generally unrecoverable.

As the soil is subjected to an increment of deviatoric stress under undrained loading, it deforms. Because of the high incompressibility of pore water a certain portion of stress increment is taken up by pore water, while the rest is shared by soil structure. When the soil is unloaded, it rebounds recovering a portion of total deformation and pore pressure reduces. At the end of unloading a residual pore pressure persists corresponding to the changed structural arrangement of soil. When the soil is reloaded to the original stress level additional deformation sets in the sample resulting in an increased peak pore water pressure. Again on unloading, as the unrecourable strain is larger than obtained after previous unloading, a higher residual pressure remains in the pore water. This built up of peak as well as residual pore water pressure and strain continues on subsequent cycling till such time when the soil structure attains an equilibirium with the cycled stress level.

Pore Pressure Versus Strain:

For each test of all the series, the pore pressure versus strain during cycling is plotted in Figs. 5-1 to 5-7.

Only the values at the end of loading and unloading are presente
The numbers of cycles are also marked on them. These plots reveal that:-

The first cycle produces much of pore pressure and strain depending on the level of cycled stress and subsequent cycles build them up gradually. This is because the larger portion of total unrecoverable deformation possible from readjustment of the soil particles at a stress level, occurs in the first cycle itself. The subsequent cycles cause only little additional readjustment.

The pore pressure - strain relationship for peak as well as residual is essentially linear after a few cycles. An initial upward concavity is observed in tests TB-2 and TC-2 (Figs. 5-3 and 5-6). The linearity of pore pressure - strain relationship results from the fact that the development of pore pressure is only a progressive strain effect for small strain increments. And during cycling at all stress levels only small strain increments are generated.

The pore pressure values at the end of loading and unloading do not fall on a unique pore pressure-strain curve. Also, during unloading pore pressure versus strain does not retrace that obtained during loading. This results from the difference in compressibility and expansibility coupled with the fact that corresponding to the same strain the structural

configuration of soil is not alike during loading and unloading.

The spacing between the peak pore pressure-strain line and residual pore pressure-strain line increases with the cycled stress level.

Slope of Pore Pressure-Strain Line:

As already discussed, the pore pressure-strain relationship for peak as well as residual becomes linear after first few cycles and that for the same cycled stress level peak pore pressure-strain line and residual pore pressure-strain line are parallel. The slope $\Delta u/\Delta\epsilon$ of these lines is indicative of rate of pore pressure development with respect to strain. The slopes of these line are plotted against normallized level of cycled stress, q_r/q_f , for all the effective confining pressure in Fig. 5-8. This is observed that the rate of pore pressure increment with strain, $\Delta u/\Delta\epsilon$ is a function of both cycled stress level, q_r/q_f and effective confining pressure. The ratio $\Delta u/\Delta\epsilon$ decreases with increasing q_r/q_f and for the same q_r/q_f, this increases with effective confining pressure.

Such a relationship between $\Delta u/\Delta\epsilon$ and q_r/q_f results from the basic nonlinearity of stress-strain and pore pressure-strain relationship. During one cycle continued loading, at smaller strain the build up of pore pressure with respect

to strain is faster than that at larger strain. Cycling from different stress levels carries the same inherent feature in it from the respective single cycle response. When the soil is subjected to a small increment of deviatoric stress, the soil not under-going a large strain is in a state in which it is more compressible and development of pore pressure with respect to strain is faster. When the soil is subjected to a larger deviatoric stress increment, the first cycle itself takes the soil to a non-linear state in which the development of pore pressure becomes slow with respect to strain.

For the same cycled stress level, q_r/q_f the increment in pore pressure (Δu) increases with the effective confining pressure at a faster rate than the increment in strain ($\Delta \epsilon$). This explains the incluence of confining pressure on the rate of development of pore pressure with respect to strain, (Fig. 5-8).

Pore Pressure and Strain Versus Number of Cycles:

The pore pressure and strain at the loading and unloading ends of cycle have been plotted against number of stress cycles applied (Figs. 5-9 to 5-16).

During cycling at all stress levels below failure stress q_f, the pore pressure and strain, both peak and residual increase with number of applied stress cycles.

However, their increments per cycle are observed to decrease with the increasing number of cycles. Though, because of time limitation an equilibirium stage was not achieved during tests, a probable extension (shown with dotted line in all the Figs. from 5-9 to 5-15) of curves reveals that the pore pressure and strain, both peak and residual attain a maximum limiting value. The equilibirium states are marked on the curves by ▼ sign. The number of cycles required to attain this equilibirium condition is a function of cycled stress level and the effective confining pressure. For the same confinement pressure, higher the level of cycled stress lesser the number of cycles required. The increase in effectiv confining pressure also reduces the number of cycles required to reach equilibirium. The following table illustrates this observation:-

TABLE I

Effective Confining Pressure (psi)	Level of Cycled Stress q_r/q_f (%)	No. of Cycles Required to Reach Equilibirium N_{eq}
5.0	36.2	46
	53.2	42
30.0	23.0	34
	46.8	28
	74.0	11
45.0	25.5	21
	52.0	13

After the equilibibirium condition is reached, further applications of stress cycles do not augment the peak pore pressure and strain. Upon unloading also, all the time same residual pore pressure and nonrecoverable strain are achieved. Thus pore pressure-strain curve retraces the same loop. After the equilibibirium is attained the soil behaves in an essentially elastic manner. For an elastic condition, theoretically, the increment in pore pressure should be equal to one-third of the increment in total axial stress. This in fact is observed to more or less true as demonstrated in the following table :-

TABLE II

(psi)	q_r/q_f (%)	q_r (psi)	U_{peak} (psi)	$U_{residual}$ (psi)	$U_{peak}-U_{residual}$ (psi)	$1/3 \times q_r$ (psi)
5	36.2	1.74	2.16	1.6	0.56	0.58
	53.2	2.55	3.20	2.20	1.00	0.85
30	23.0	4.42	6.55	5.05	1.50	1.47
	46.8	9.00	10.20	7.25	2.95	3.00
	74.0	14.20	16.80	12.00	4.80	4.73
45	25.6	7.24	7.00	4.50	2.50	2.42
	52.0	14.74	16.50	11.50	5.00	4.92

5.3 Critical Level of Stress for Repeated Loading, Equilibirium Lines :

There exists a limit to which the pore pressure and strain can be increased by continued application of a small cyclic stress, after which an equilibirium is attained.

The equilibirium pore pressure and equilibirium strain are functions of cycled stress level, q_r/q_f and the effective confinement pressure, σ_c' . The equilibirium pore pressure , U_{eq} and the equilibirium strain, ϵ_{eq} are plotted against the normallized cycled stress level, q_r/q_f in Figs. 5-17 and 5-18. A linear relationship is observed between the equilibirium pore pressure and the cycled stress level. And slope of the equilibirium pore pressure-cycled stress level is observed to increase with the confining pressure.

The total strain developed till the equilibirium condition, increases with the cycled stress level but at an increasing rate. However for the same cycled stress level, the equilibirium strain is reduced by increase in effective confining pressure.

On the stress space plot already presented (Figs. 4-4 4-6 and 4-20 etc.) it is observed that with the repeated application of stress, the stress path migrates towards K_f-line. But the observed equilibirium condition restricts

this migration to a limit and therefore it is not possible to cause failure by the repeated application of the original stress level. All such equilibirium states of stresses corresponding to the loading end of different cycled stress levels, when plotted on stress space, constitute a line (Figs. 4-6 , 4-14 and 4-20). This line is termed as equilibirium line (Sangrey, 1968). The intersection of the this line with the K_f line defines a critical level of stress for repeated loading, below which continued cycling will always result in an equilibirium condition without reaching failure and above which repetition of the stress will build up enough pore pressure to casue failure (Figs. 4-4 and 4-6). This is observed that the inclination of equilibirium lines is more or less independent of the effective confining pressure ie. they are parallel. Sangrey (1968) observed that the inclination of these equilibirium lines increases with overconsolidation ratio.

Effect of Confining Pressure on the Critical Stress Level for Repeated Loading:

The critical stress, q_{cr} for different effective confining pressure, after being normallized by respective one cycle failure stress, q_f, are plotted against the effective confining pressure , σ_c' (Fig. 5-19). The extension of the curve (shown with the dotted line) reveals that in lower ranges of confining pressures, the critical stress level, q_{cr}/q_f is very low but increases rapidly with the increase in effective confining pressure. However, in higher ranges of the confining pressure, this increase becomes slow and finally, the critical stress level, q_{cr}/q_f tends to 100 % line asymptotically.

The $q_{cr}/q_f - \sigma_c'$ point for the soil used by Sangrey (1968) has also been plotted in Fig. 5-19 . He had only one effective confining pressure. However, a comparison reveals that for the same effective confining pressure σ_c' , the critical stress level for repeated loading q_{cr}/q_f is low for lower PI soil. This is expected to be true for all ranges of effective confining pressure. Therefore a less plastic soil is expected to be more prone to failure under repeated loading, other conditions remaining the same.

Mechanistic Picture:

Each repeated application of a stress changes the structural arrangement of soil skeleton imparting more and more rigidity to it. Therefore, during a sequence of repeated application of a stress, a cycle produces a lesser change in structural configuration than produced by the preceeding cycl This results in decrease of per cycle increment in strain and pore pressure. Ultimately, a stage is reached in which no further structural rearrangement is possible and the structural configuration attains an equilibirium with the applied repetetive stress. After such an equilibirium state is attained, further build up of pore pressure and strain stops.

For lower cycled stress level, the total amount of pore pressure required to reach failur is compartively larger than that required at higher cycled stress level. This fact becomes very clear when one observes the reducing distance between one-cycle failure stress path and the K_f -line. Now, depending upon the rate of change in structural configuration, there exists a level below which repeated application of the stress, produces an equilibirium condition before the total amount of pore pressure required to cause failure is develope

However, above this level the cyclic application of stress builds up enough pore pressure to cause failure before reaching any equilibirium state.

5.4 Post Cycling Behaviour:

In all the tests, after application of cyclic stress, the sample was loaded to failure or at least close to failure After cycling at a pre-failure stress level the samples experience increased pore pressure resulting in overconsolidation. The shape of stress path during loading subsequent to cycling depends upon the degree of overconsolidation induced. For the values of overconsolidation ratio, (n) of 1.15 or less , the stress path merges smoothly with single-cycle loading stress path (Figs. 4-10 and 4-18) and follows it till failure.

For values of overconsolidation ratio (n) between 1.15 and 1.4, the stress path rises vertically and than merges with single cycle loading stress path near the failure stress level (Figs. 4-12 and 4-20). However, for overconsolidation ratio (n) between 1.4 and 2, the stress path rises vertically to meet single-cycle failure point C (Figs. 4-6 and 4-14).

This observation is in accordance with the findings of Wroth and Loudon (1967).

5.5 Post-Failure Behaviour:

When the soil is subjected to failure stresses repeatedly, deviation in failure obliquity value was observed with almost no change in undrained strength, for the low confining pressure (Fig. 4-2). However, for lorger confining pressures, the subsequent failure stresses followed the same K_f-line resulting in reduced undrained strength but a constant failure obliquity (Figs. 4-8 and 4-16). According to Sangrey (1968) continued cycling to failure will bring down the failure stress to critical stress level q_{cr} and than an equilibirium state will be attained.

During cycling to failure stress level the rate of pore pressure development with respect to strain, $\Delta U/\Delta \epsilon$ is very low. This is because microscopically well defined slip planes are developed once failure is reached.

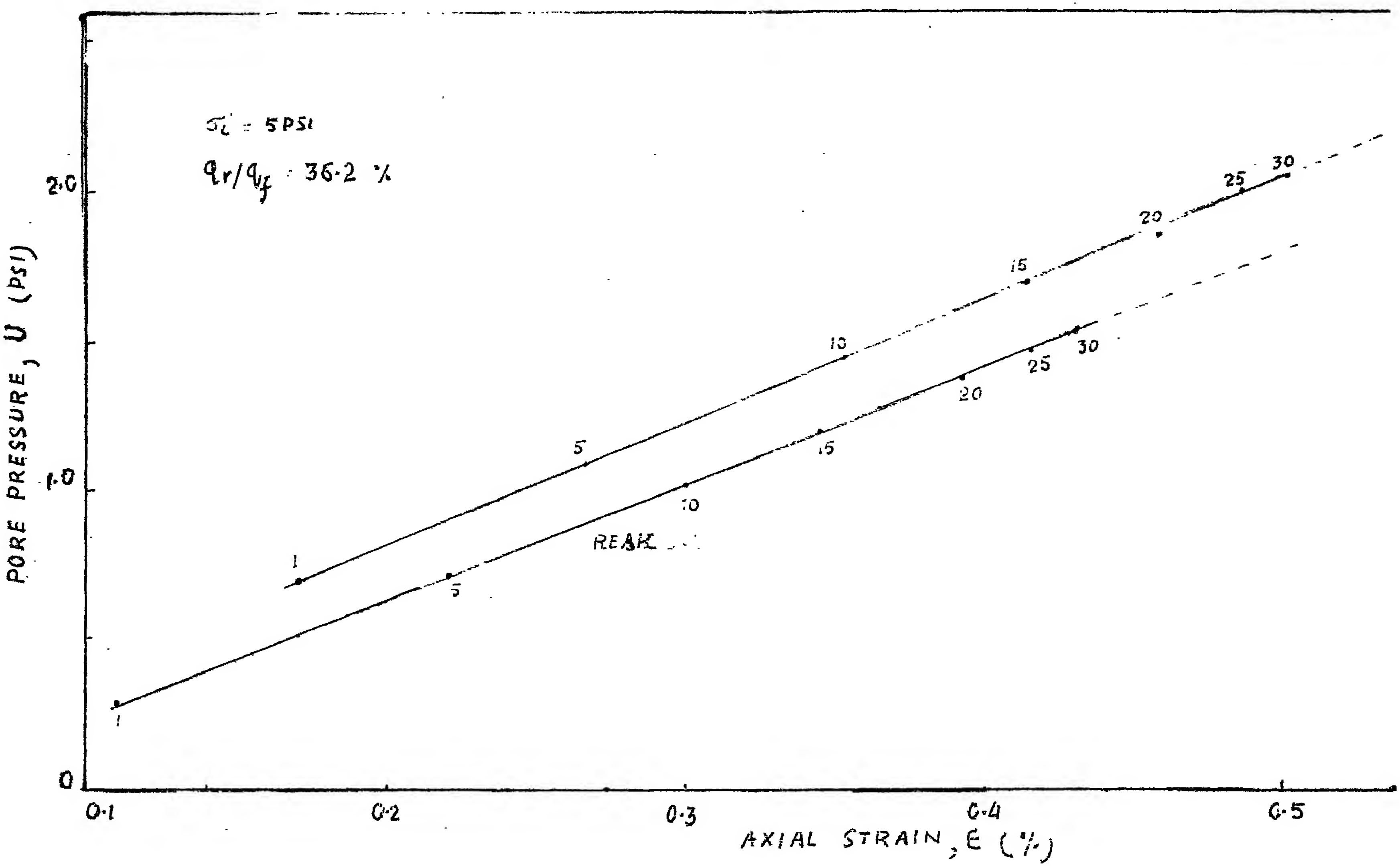

Fig. 5-1 Pore Pressure - Strain During Cycling - Test TA-2.

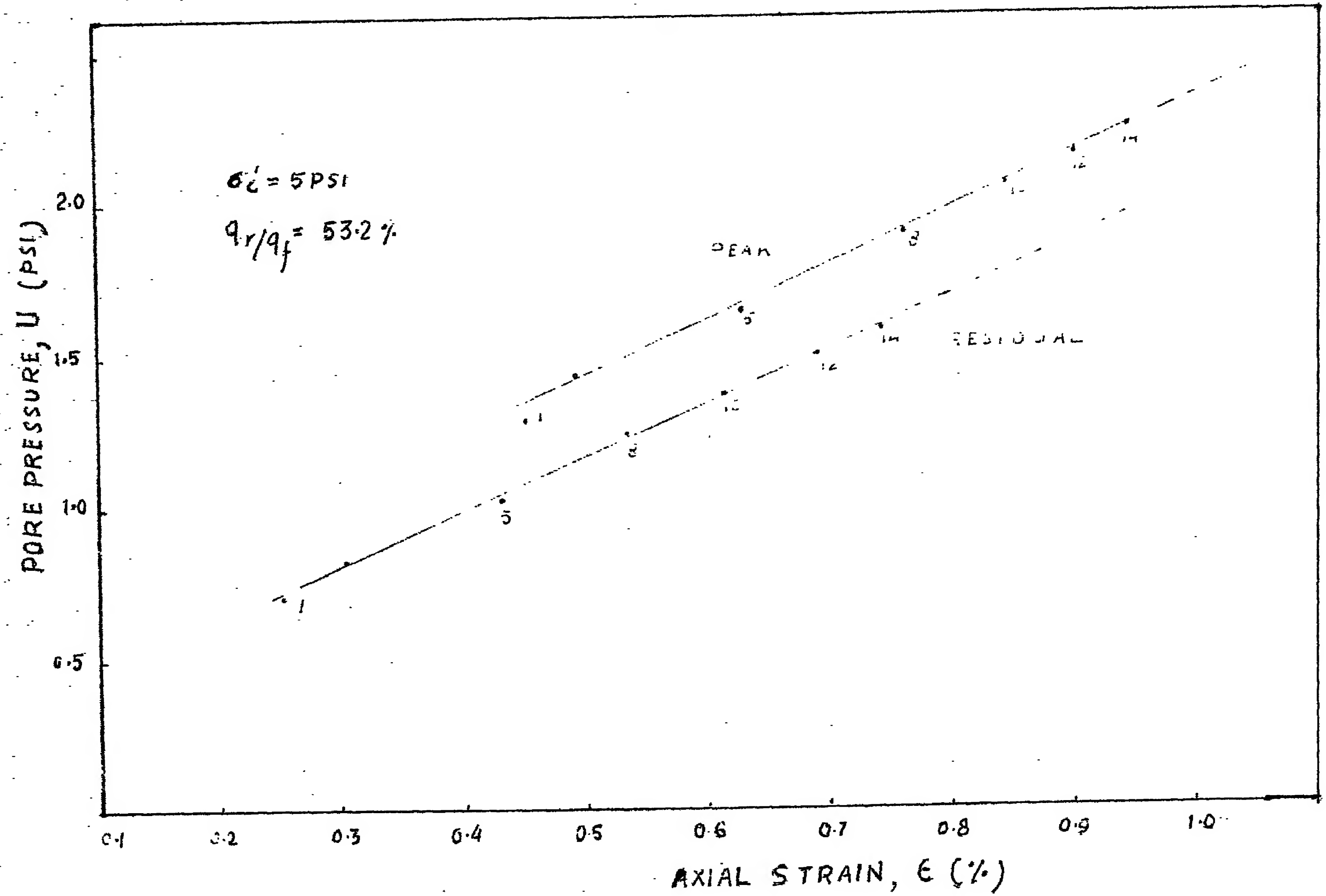

Fig. 5-2. Pore Pressure - Strain During Cycling - Test TA-3.

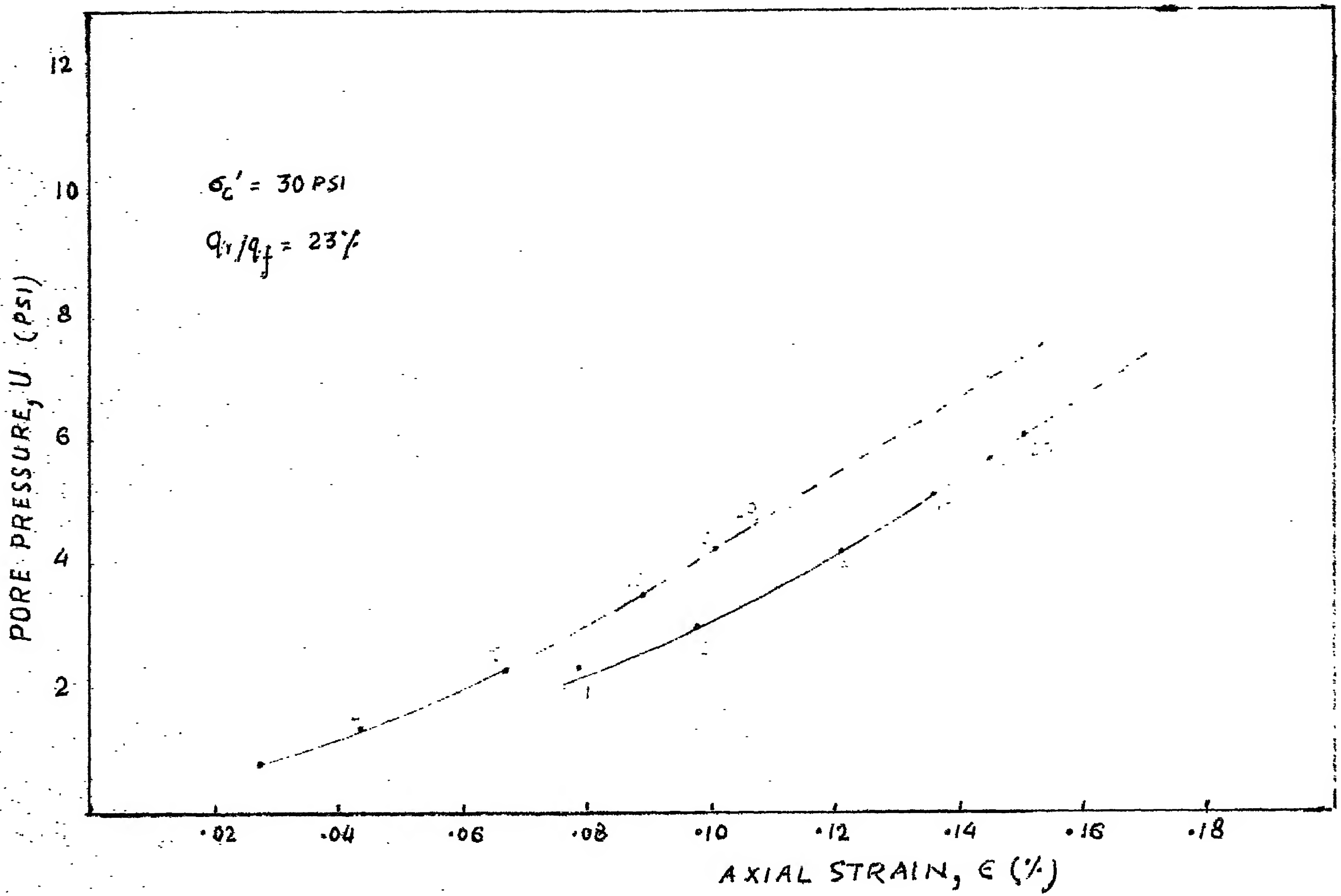

Fig. 5-3 Pore Pressure — Strain During Cycling - Test, TB-2

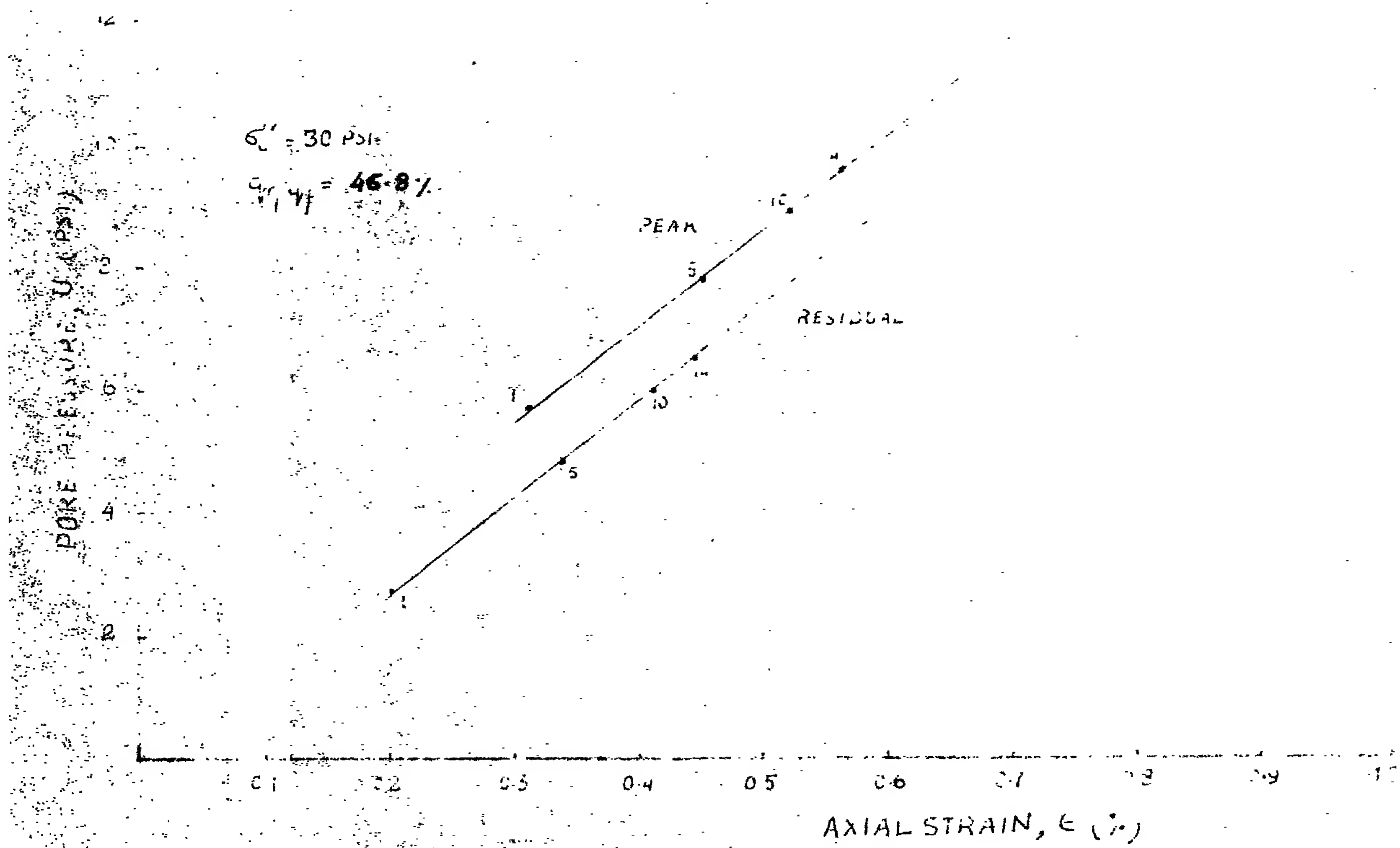

Fig. 5-4. Pore Pressure - Strain During Cycling – Test TB-3.

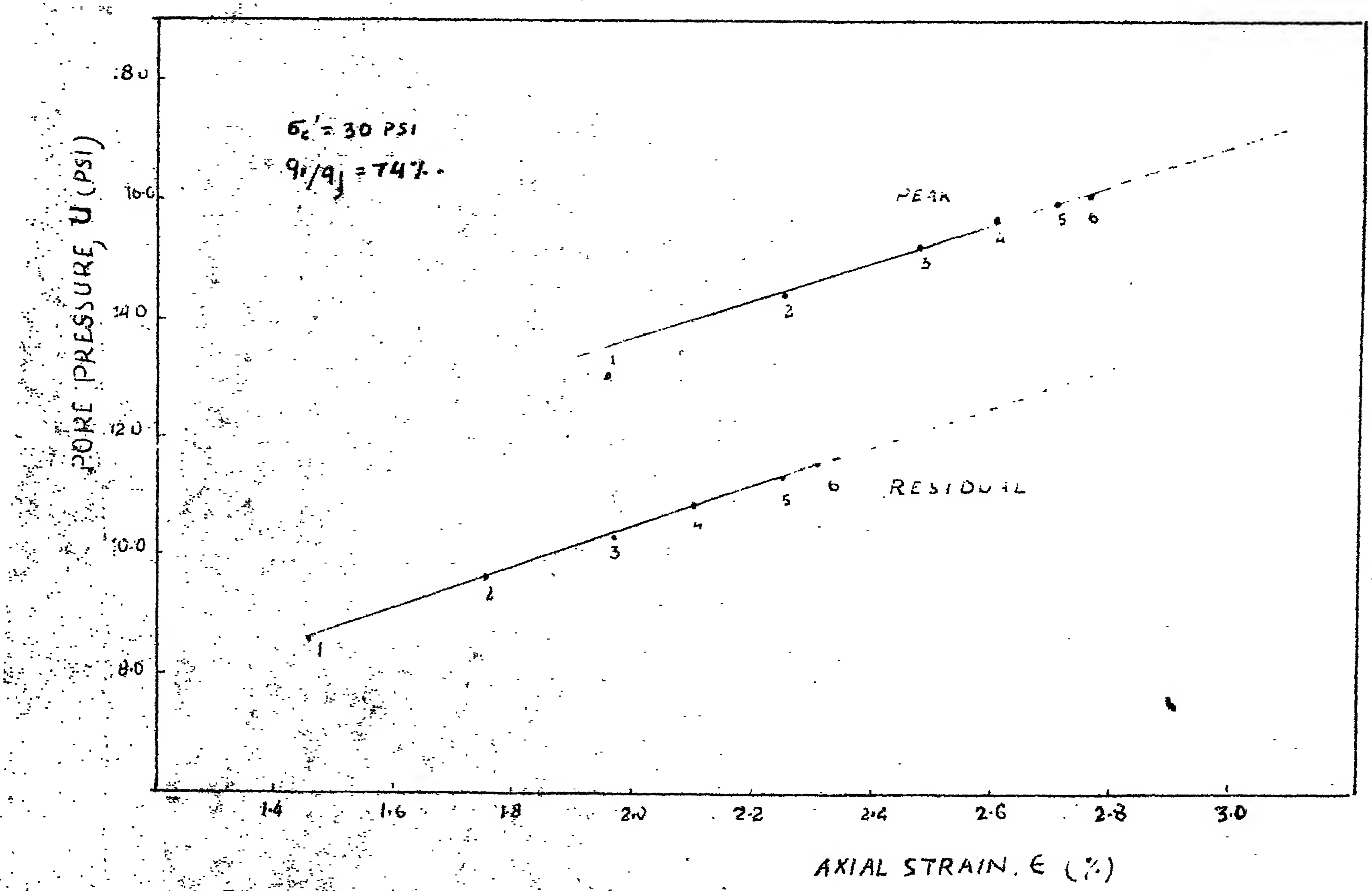

Fig. 5-5. Pore Pressure - Strain During Cycling - Test TC-4.

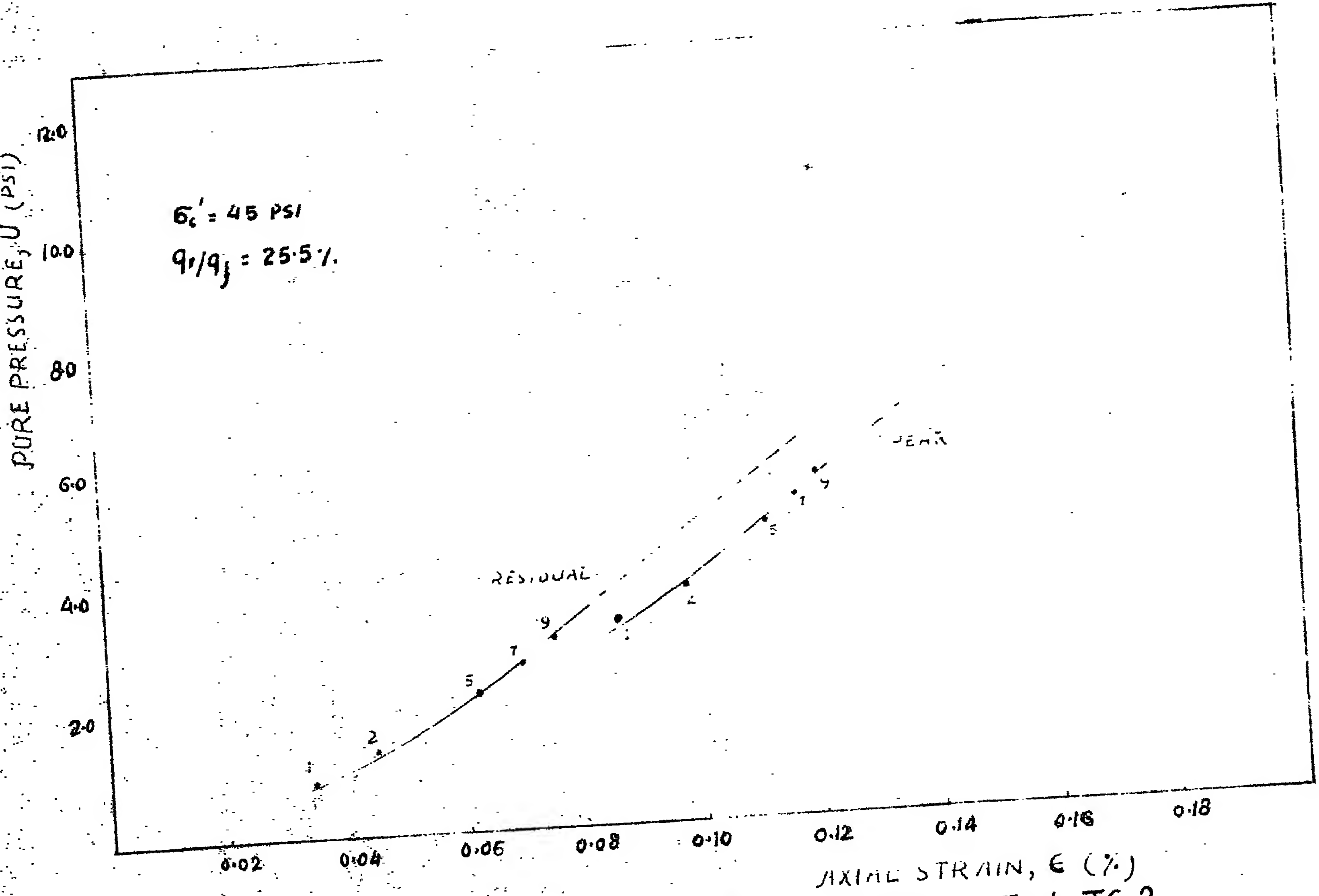

Fig. 5.6: Pore Pressure - Strain During Cycling - Test TC-2.

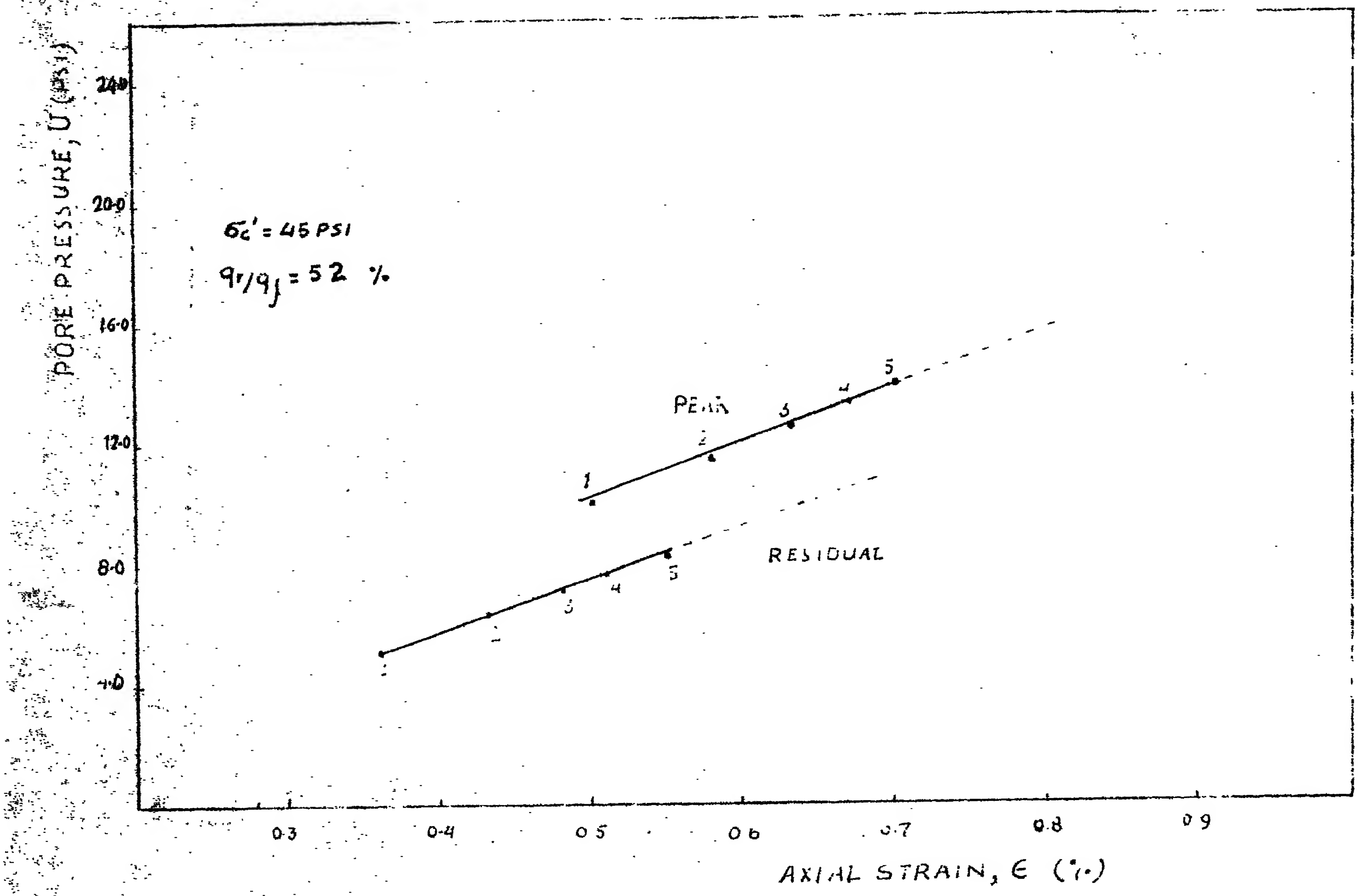

Fig. 5-7. Pore Pressure - Strain During Cycling - Test, TC-3.

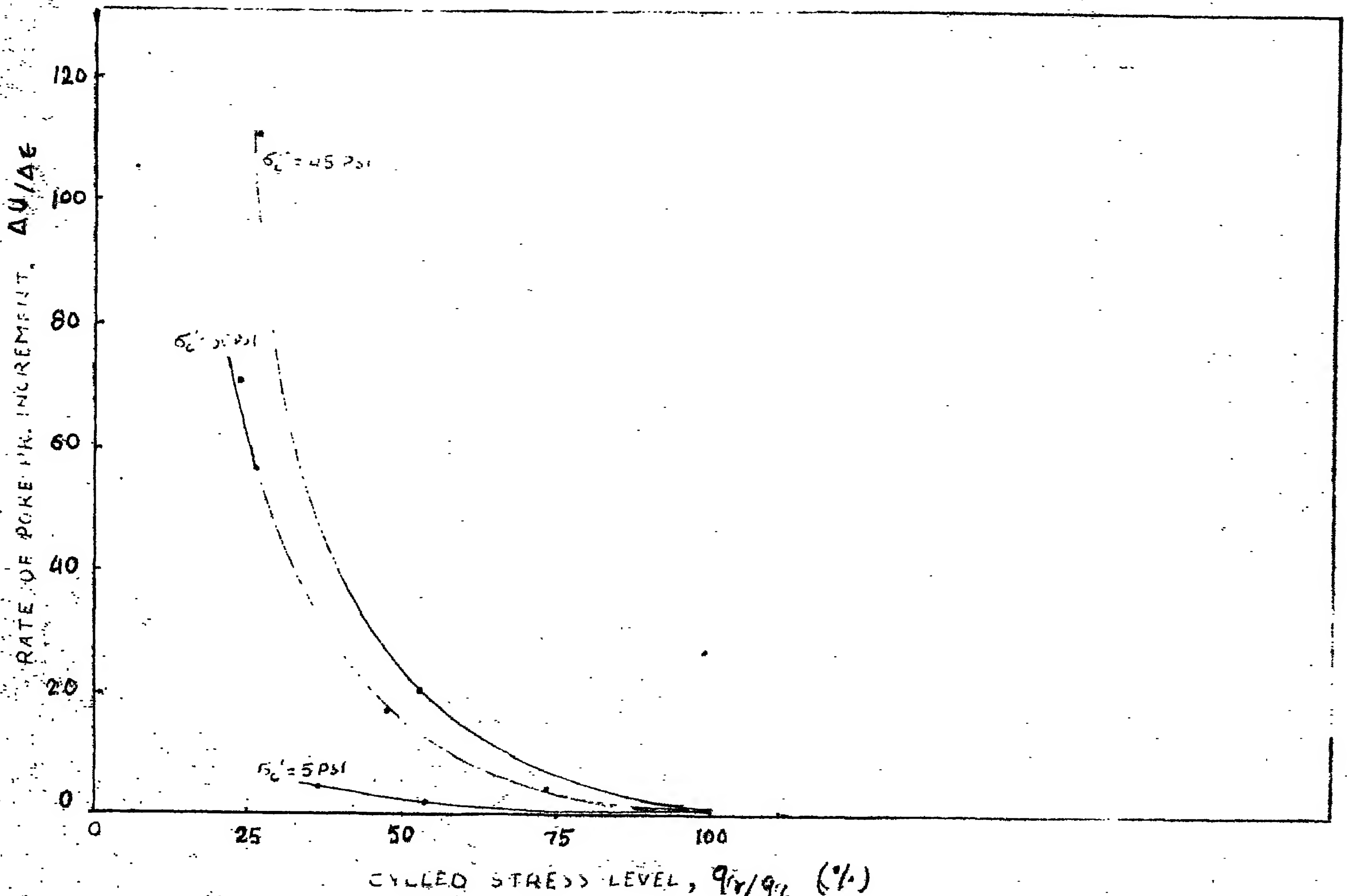

Fig. 5-8. Relationship Between Rate Of Pore Pressure Developement ($\Delta U/\Delta \epsilon$) And level Of Cycled Stress (q_r/q_f).

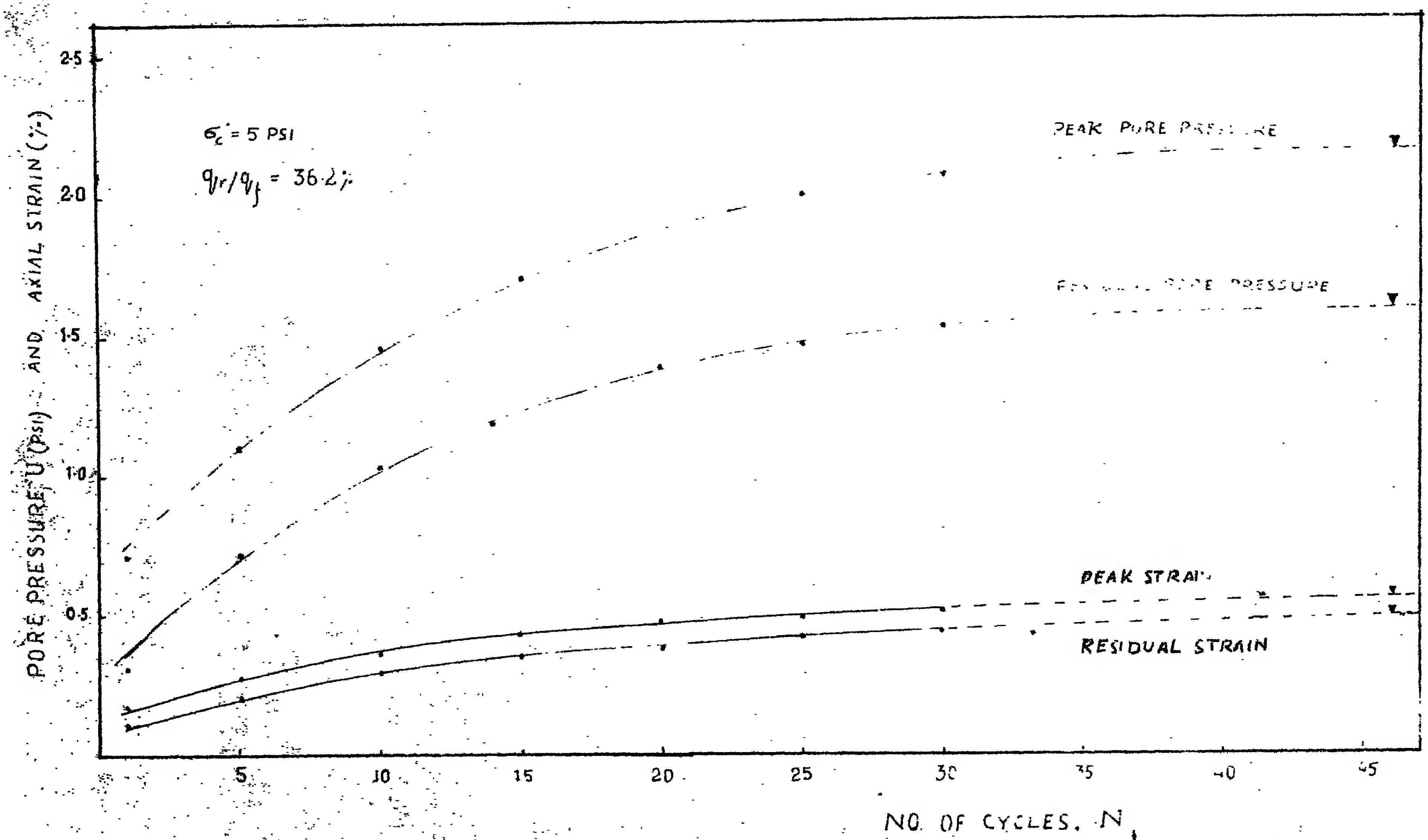

Fig. 5-9. Pore Pressure And Strain Versus N... ...cles

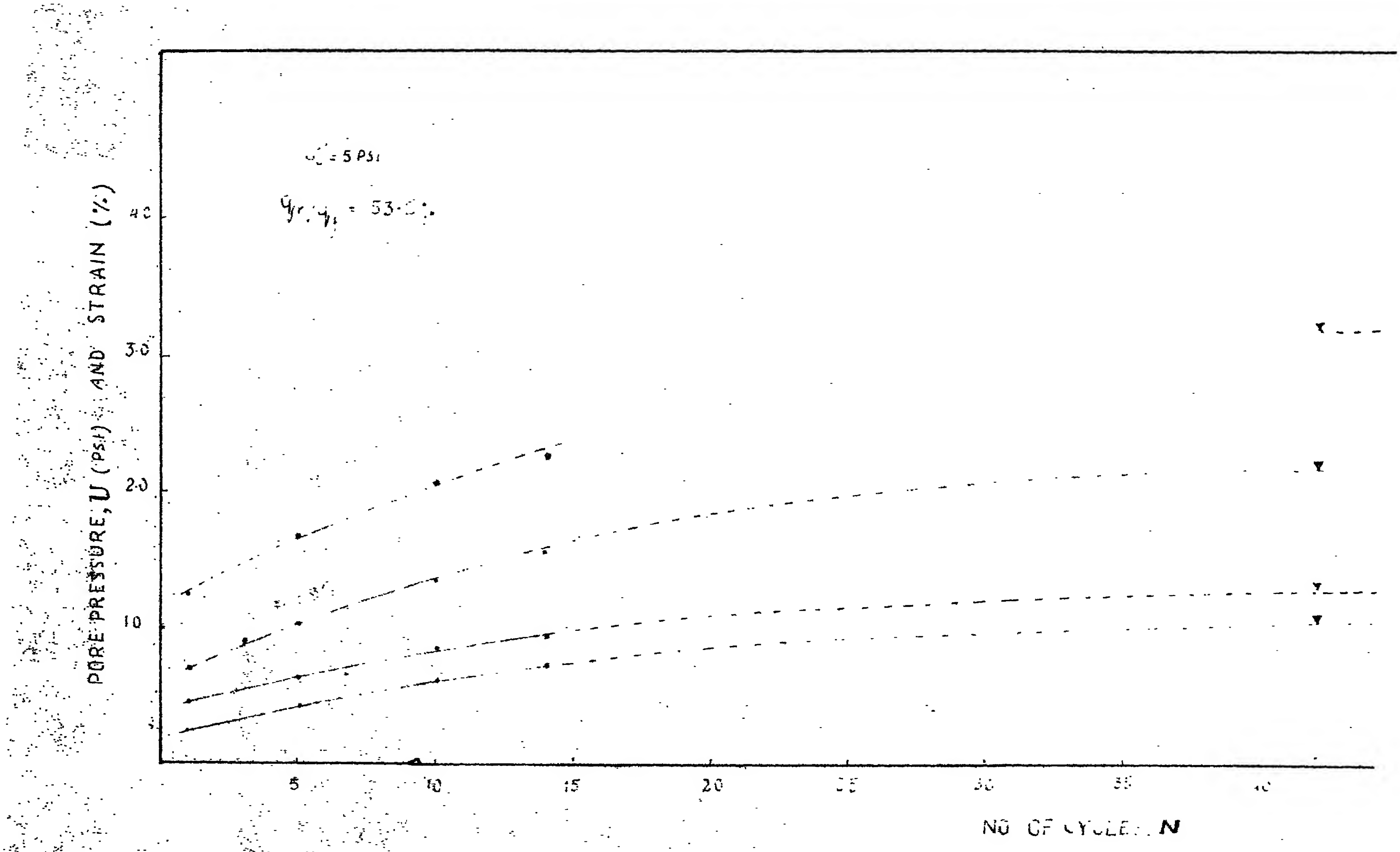

Fig. 5-10. Pore Pressure And Strain Versus Number of Stress Cycles

Test TS-3

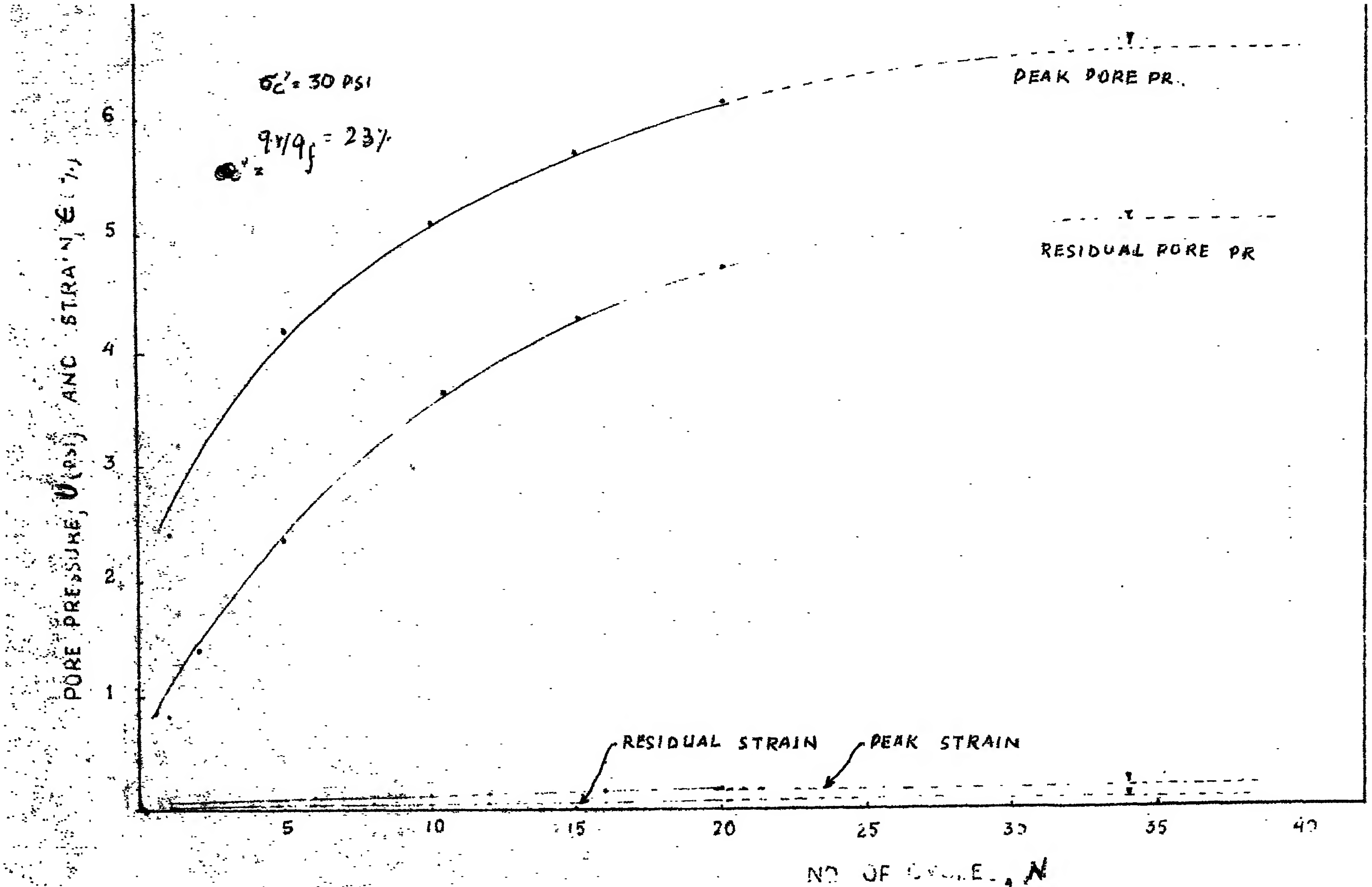

Fig. 5-11. Pore Pressure And Strain Versus Number Of Strees Cycles

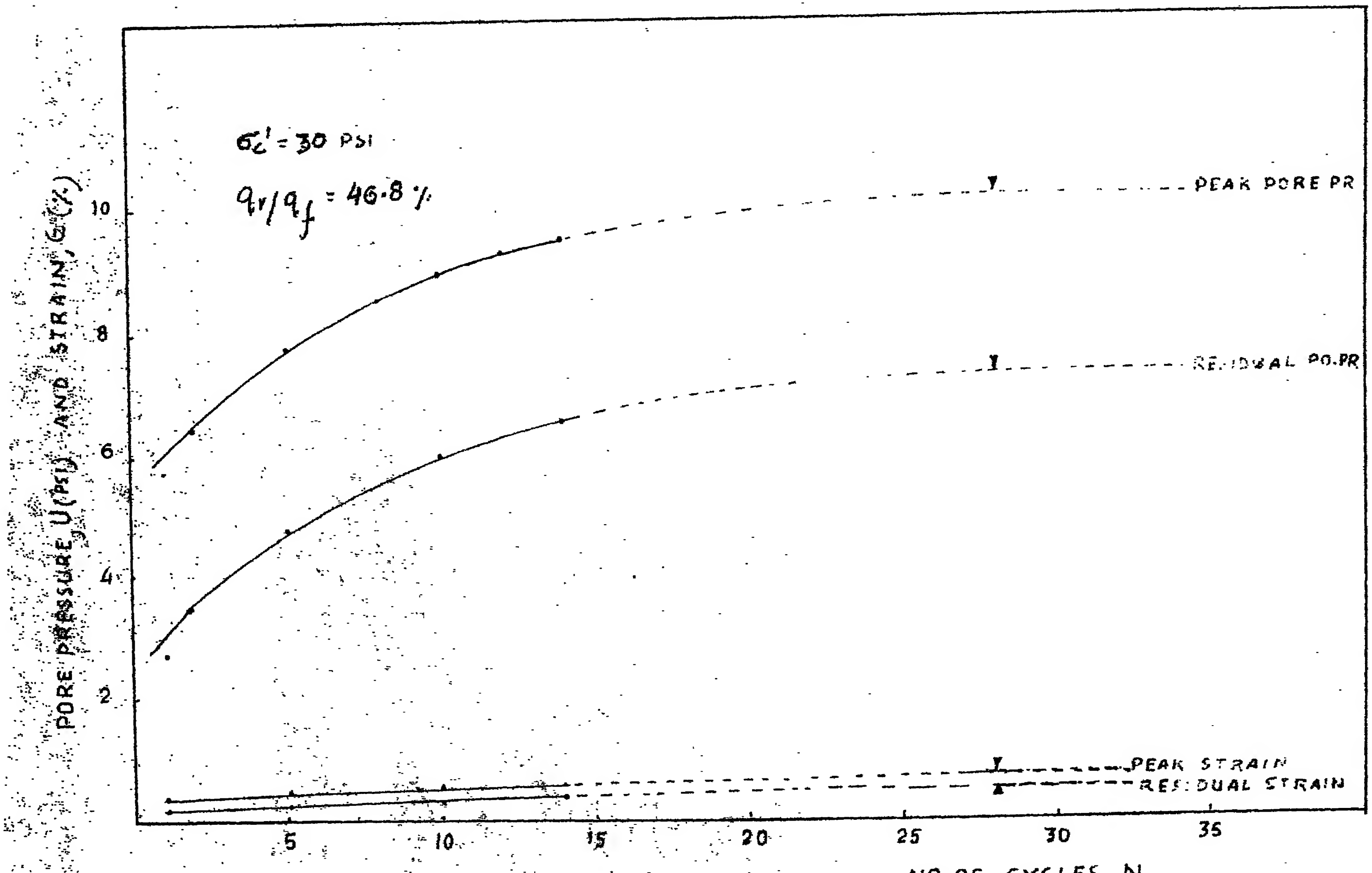

Fig. 5-12 Pore Pressure And Strain Versus Number Of Stress Cycles
Test, TB-3.

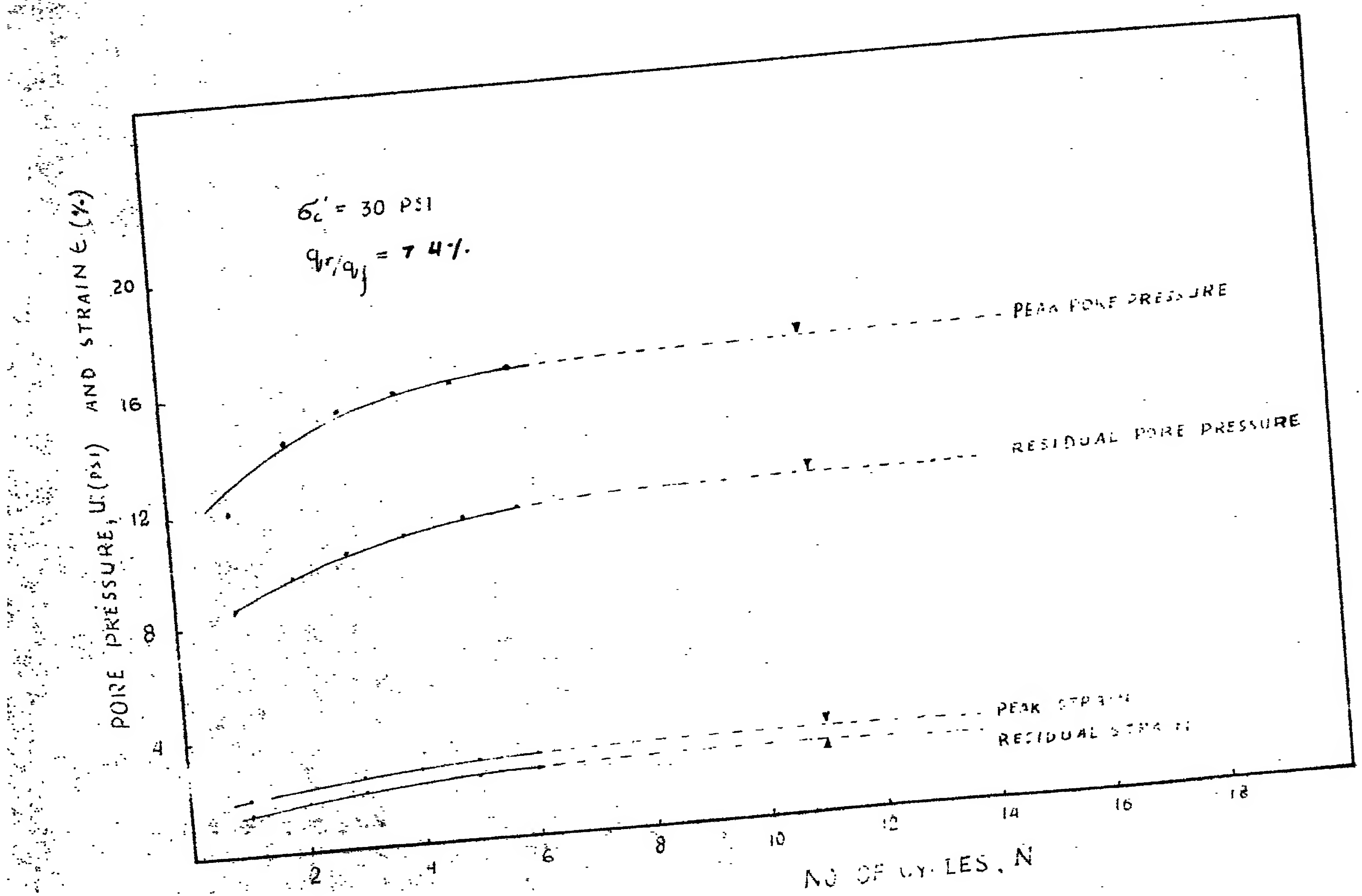

Fig 5-13. Pore Pressure And Strain Versus Number Of Stress Cycles
Test TC-4.

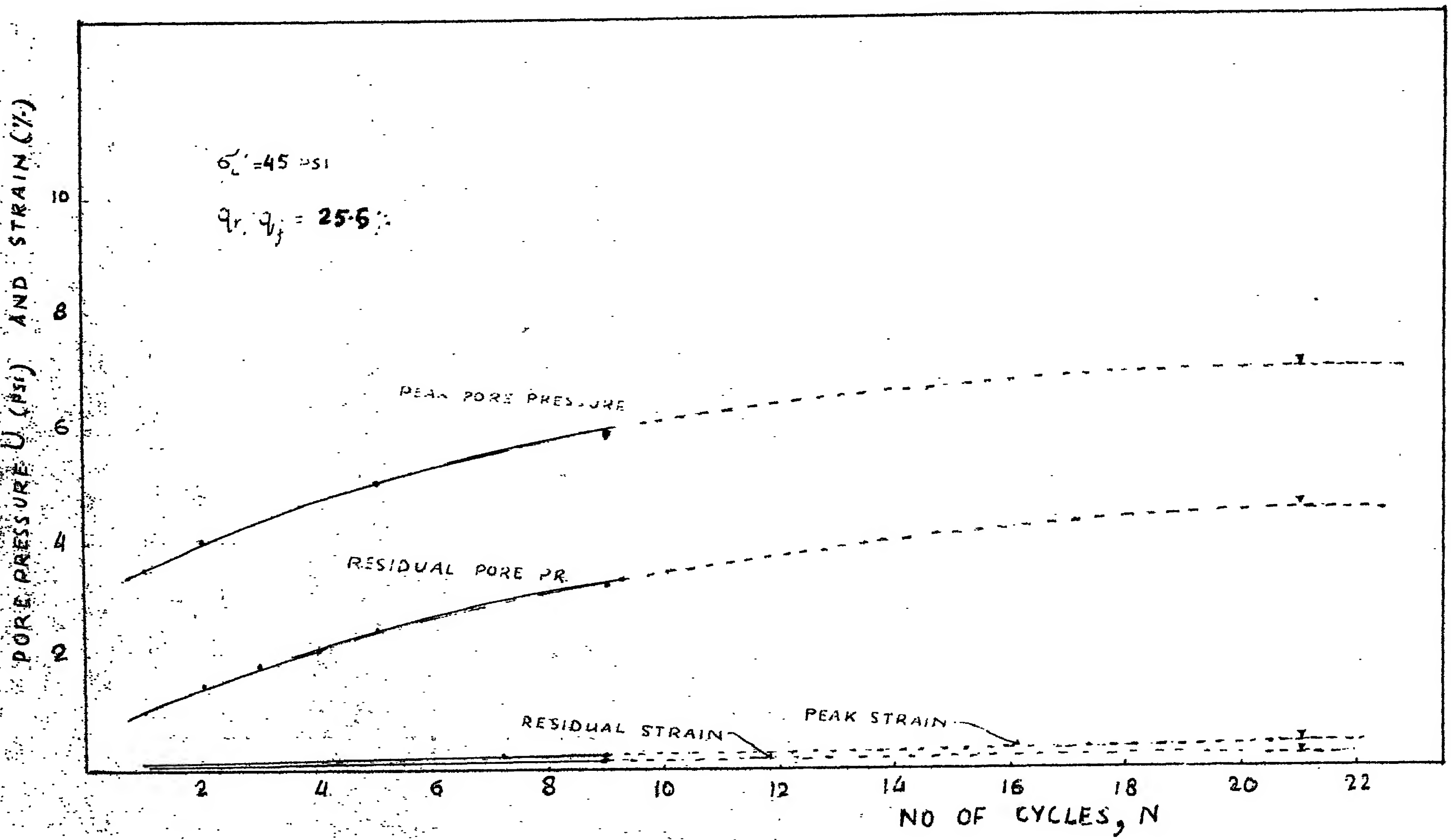

Fig. 5-14. Pore Pressure And Strain Versus Number Of Stress Cycles.

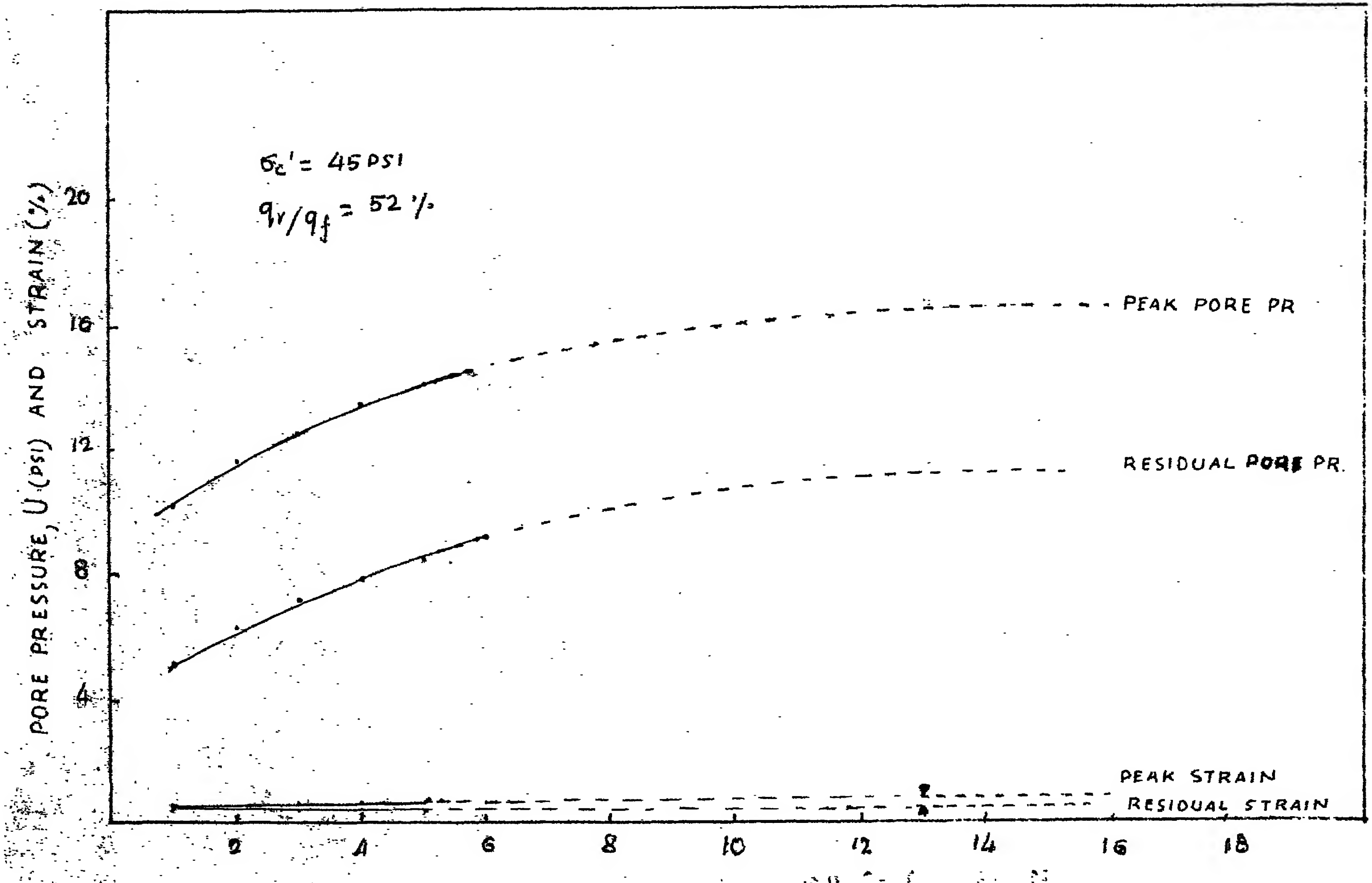

Fig. 5-15. Pore Pressure And Strain Versus Number Of Cycles

Test TC-3.

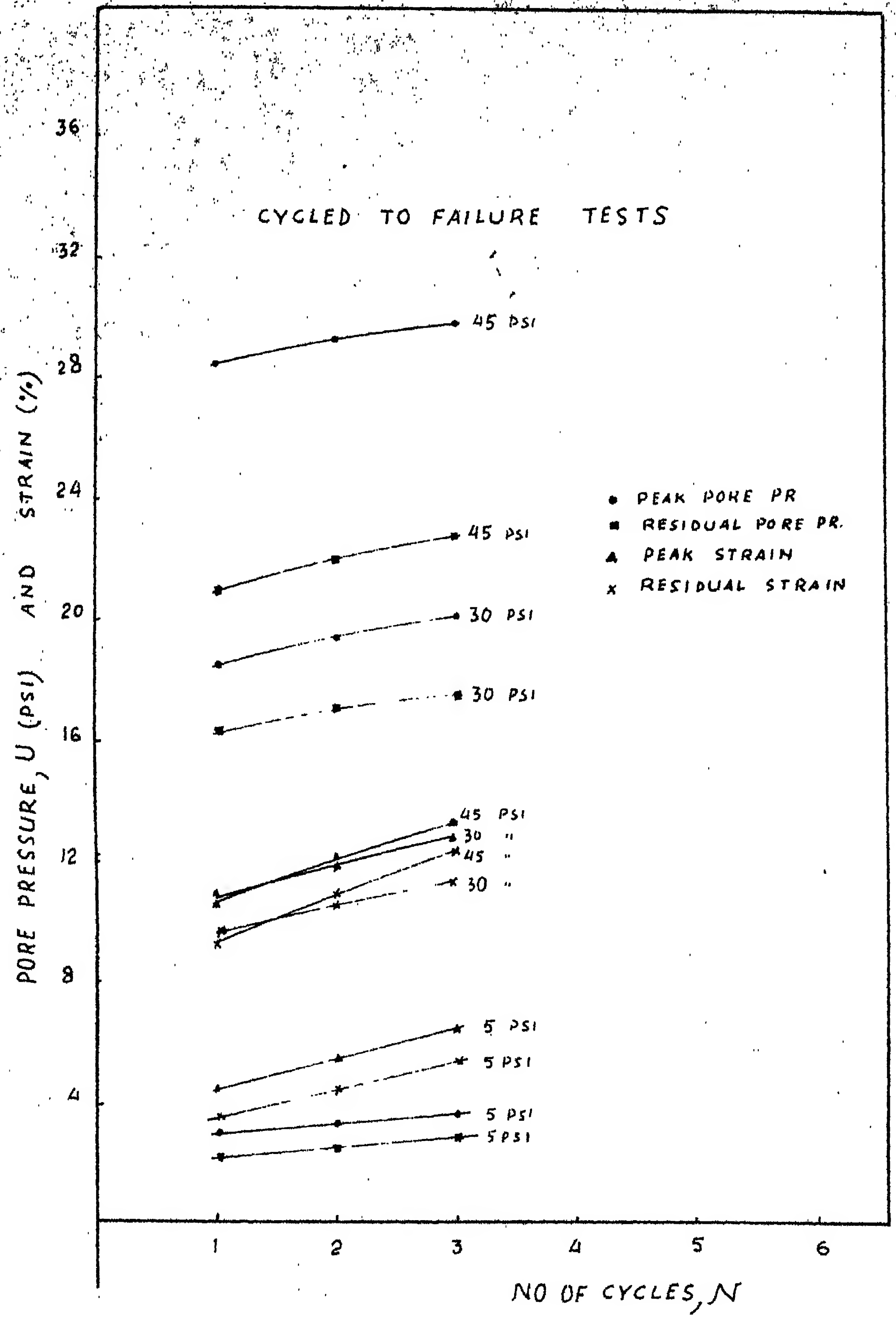

Fig. 5-16. Pore Pressure And Strain Versus Number Of Cycles Test TA-1

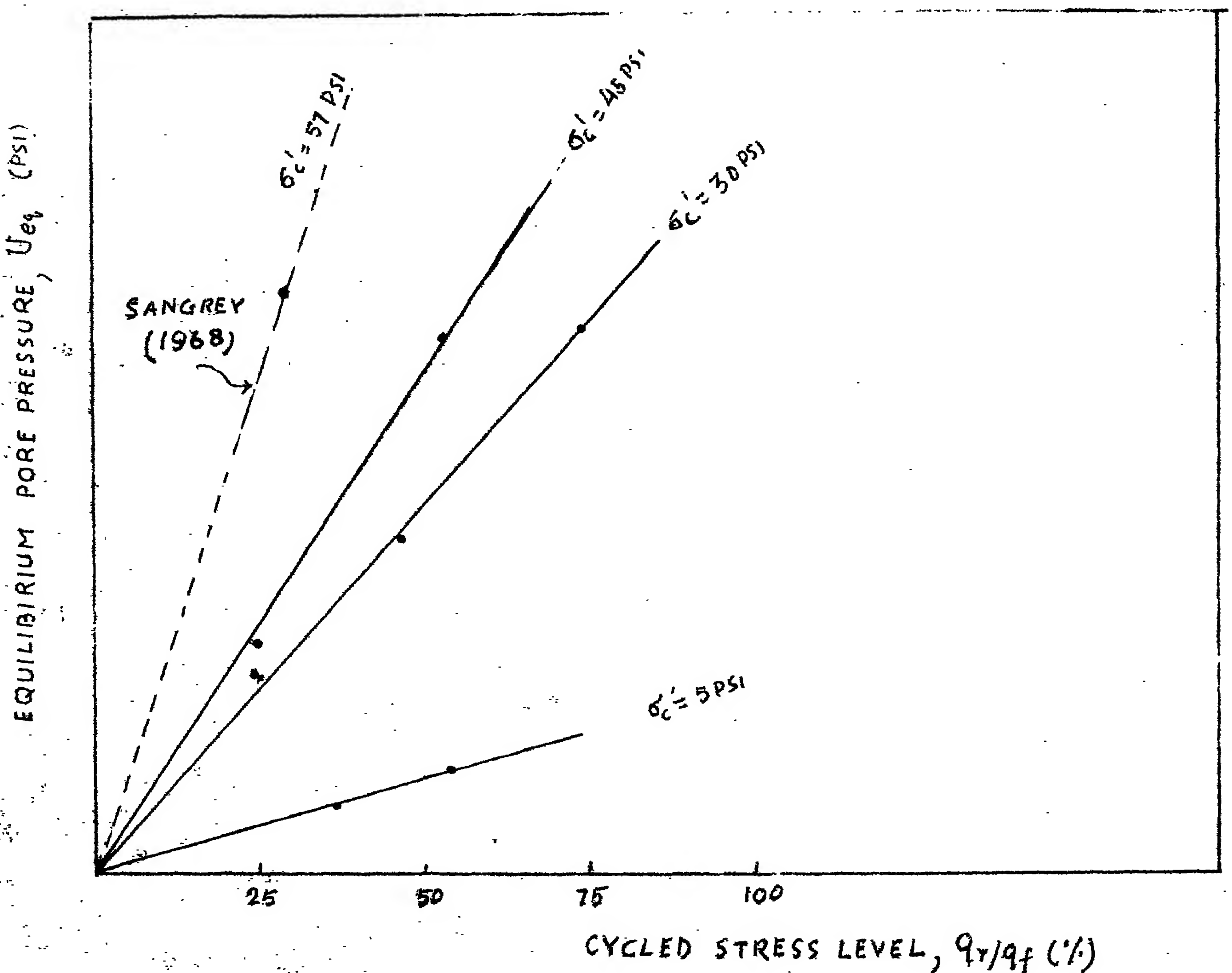

Fig. 5-17. Relationship Between Equilibrium Pore Pressure

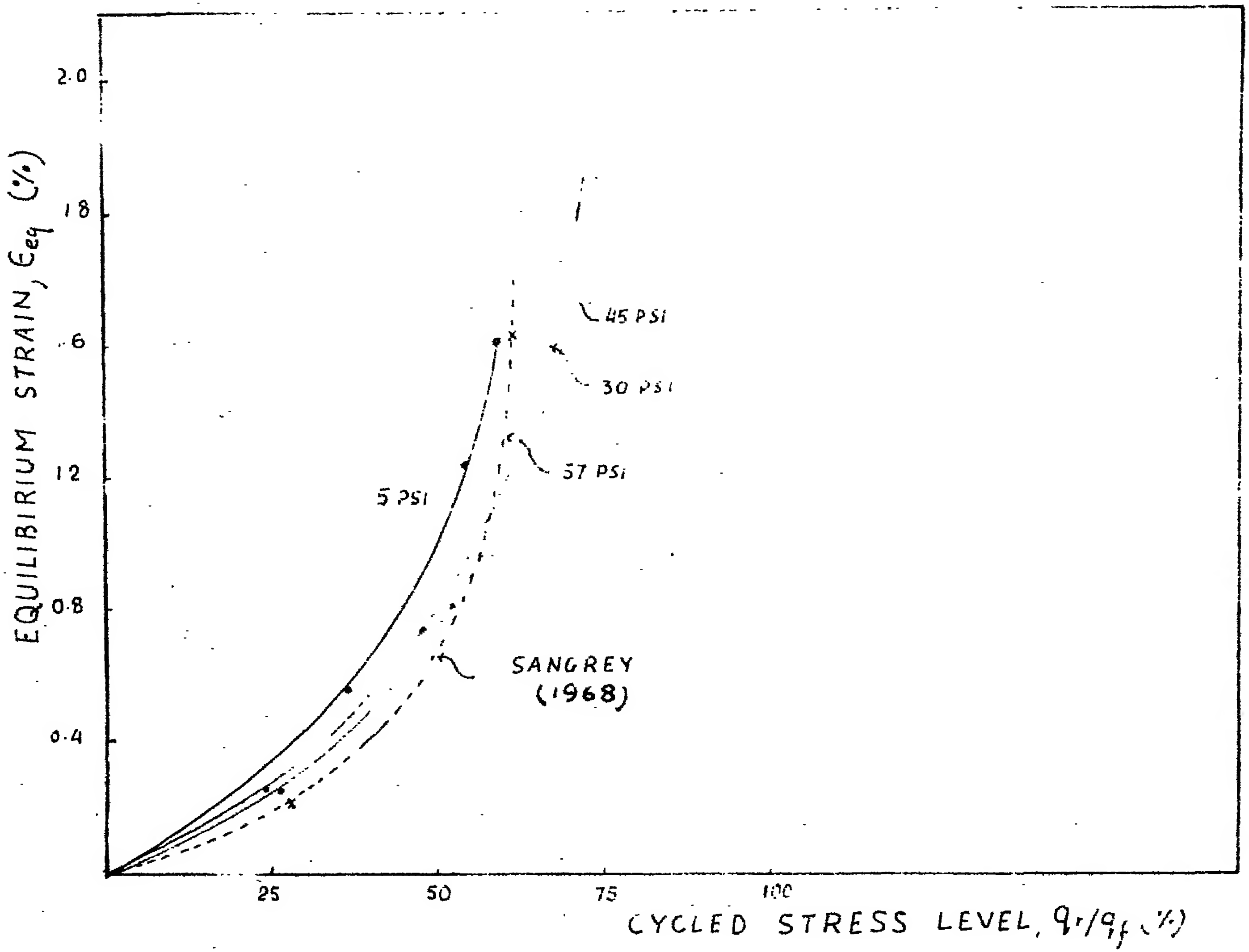

Fig. 5-18. Relationship Between Equilibrium Strain And Level Of Cycled Stress.

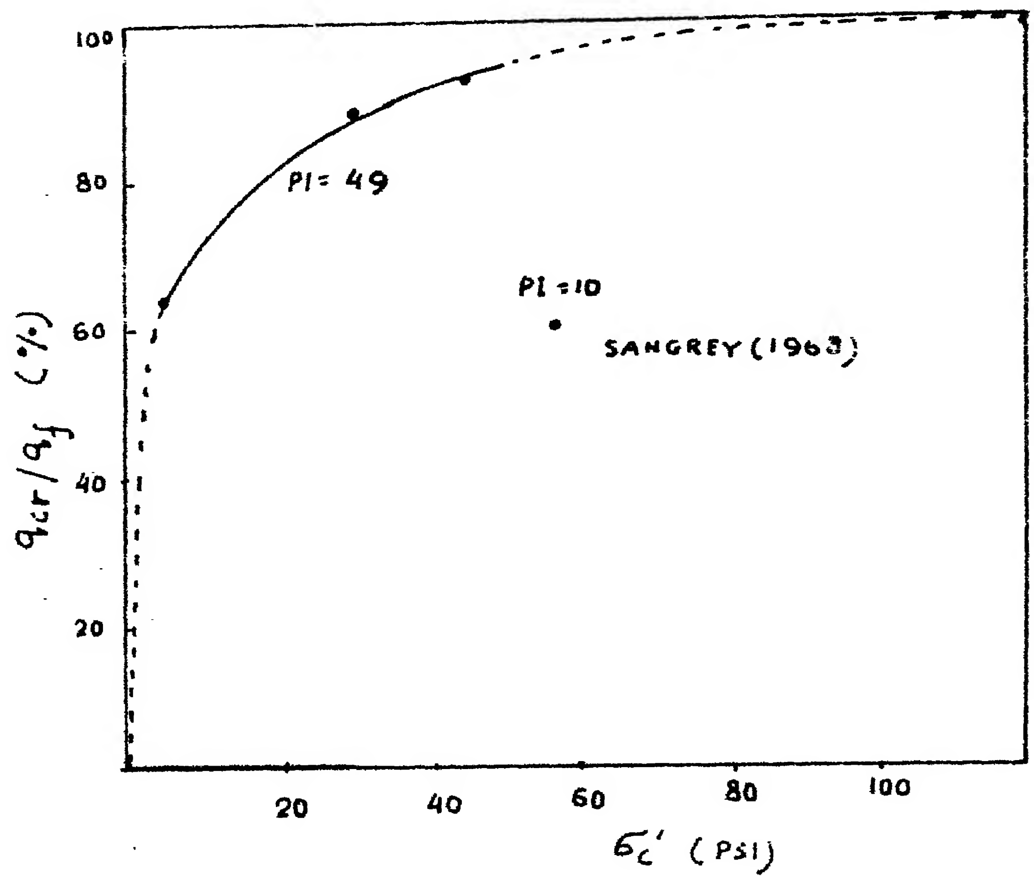

Fig. 5-19. Effect Of Effective Confinement Pressure On Normallized Critical Level Of Cycled Stress.

CHAPTER VI

CLOSURE

6.1 Conclusions:

The response of saturated soil to repeated loading is markedly different from that to a single load application. This is a consequence of pore water pressures which are built up during load repetition. The magnitude and rate of pore pressure build up are dependent upon the effective confining pressure and the level of repeated stress. An increase in axial stress followed by a removal of the stress increment results in residual deformation and pore water pressure. Subsequent cycles of loading increase the pore pressure and deformation.

The relationship between the pore pressure, peak as well as residual and strain, during cycling is essentially linear. The rate of pore pressure development with respect to strain, increases with the effective confining pressure and decreases with the level of cycled stress.

The per cycle increments in pore pressure and deformation decrease with number of applied stress cycles because of increasing rigidity of structural configuration of soil. This, depending upon the level of cycled stress, may lead to an equilibirium condition after which the soil

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behaves in an essentially elastic manner —— the increment of pore pressure in one cycle being equal to one third of the applied axial stress.

The stress conditions corresponding to the peak loading of the equilibirium cycle can be plotted as a point in a stress space. The equilibirium points for different levels of repeated stress correponding to the same effective confining pressure, define a unique line in the stress space. This results from the linear relationship between equilibirium pore pressure and cycled stress. This line,originating at the point of consolidation pressure and terminating at the failure obliquity line, is termed as the equilibirium line, since it represents a limiting contour of elastic equilibirium. The inclination of the equilibirium line is independent of the effective confining pressure.

There exists a critical level of stress for soils subjected to repeated loading which is define by the intersection of the equilibirium line and the failure of obliquity line. Above this critical level, but below the peak strength for one cycle failure test, the repeated application of the same stress results in a continued build up of pore pressure until failure is achieved. Repeated loading below the critical stresslevel however, leads to an equilibirium condition without building enough pore pressure to cause failure

at the particular level of repeated stress.

The critical level of stress for repeated loading is function of the effective confining pressure. For small confining pressures the normallized critical level of cylic stress, q_{cr}/q_f is low and initially increases rapidly with the confining pressure. Finally, q_{cr}/q_f slowly tends to unity. This implies that at high confining pressures the repetition of load appreciably less than failure stress can not cause failure at the same stress level. However, in case of low confining pressures repeated load considerably less than the failure strength may induce pore poressures during cycling high enough to cause failure.

For the same confining pressure, low PI clay is more prone to failure at stresses less than maximum deviatoric stress due to cycling as compared to highly plastic clay.

Pre-failure cycling induces overconsolidation in the as the increased pore pressures decreases the effective stresses. The shape of stress paths during loading subsequent to pre-failure cycling depends upon the degree of over-consolidation induced in the soil.

6.2 Relevance:

Frequent landslides have been reported to occur in soft marine sediment in deltaic regions of many rivers. These sediments have a loose structure and due to the large degree of underconsolidation resulting from fast rate of sedimentation, they are subject to low confining stresses. According to the observation made in this study, the critical level of repeated stress, q_{cr} for such a material will be much below the failure stress q_f. Such sediments are subjected to the repeated loading of waves. Even moderate waves are expected to produce a stress level higher than the critical which will cause failure because of continued build up of pore pressures.

However, for the type of plastic clay used in this study, in most of the situations with high confining pressure acting, the failure under repeated loading will not be of concern, q_{cr} being very close to q_f. Nevertheless, a knowledge of pore pressure generation after repeated application of stress will help a great deal in analysing the settlements caused by dissipation of excess pore pressure induced due to cycling of stress. Such a settlement analysis is very much called for various engineering structures for example silos and oil tanks in which emptying and filling produce a load of repetitive nature.

6.3 Suggestions for Further Studies:

Additional studies using different soils should be carried out to determine how general the behavioral response is, which has been observed in this study.

The difference between the critical stress level for repeated loading and the one cycle failure stress is believed to depend mainly on the plasticity of soil and its original stress history. Therefore, the author suggests that the whole range of soils having low plasticity to very high plasticity should be investigated.

An investigation into the effects of overconsolidation and initial state of stresses produced by anisotropic consolidation on the general behavioral response of soils subjected to repeated loading is considered to be important.

There may be a number of implications for the critical state approach to soils subjected to repeated loading. The relationship between the ends of equilibirium line and critical state requires a detailed study. There may emarge a possibility that the equilibirium line represents rather a fundamental lower boundary to undrained stress path and that it may define a distinct boundary between completely elastic and partially elastic behaviour.

It is expected that behavioural response to high frequency repeated load will be different than those observed in this study. Where a means for making realistic measurement of pore pressures generated under such dynamic conditions are available, a study in this direction will be useful especially for clays.

BIBLIOGRAPHY

1. Bishop, A.W. and Henkel, D.J. (1953) : "Pore Pressure Changes during Shear in Two Undisturbed Clays" 3rd ICSM and FE V. 1:94.

2. Bishop, A.W. and Henkel, D.J. (1962): The Measurement of Soil Properties in the Triaxial Test Edward Arnold; London.

3. Buchanan, S.J. and Khuri, F.I. (1954): "Elastic and Plastic Properties of Soils and their Influence on the Continuous Support of Rigid Pavements" a report to the Office of the Chief of Engineers, Military Construction and the Rigid Pavement Laboratory of the Ohio River Division Laboratories, Mariemont, Ohio.

4. Casagrande, A. and Wilson, S.D. (1951): "Effect of Rate of Loading on the Strength of Clays and Shales at Constant Water Content" Geotechnique V. 2:251.

5. Gibson and Henkel, D.J. (1954): "Influence of Duration of Tests at Constant Rate of Strain on Measured Drained Strength" Geotechnique V. 4:6.

6. Gordon, B.B. (1959): "Repeated Plate Loading Tests on Compacted Subgrades" ASTM Spec. Pub. No. 254.

7. Henkel, D.J. (1953): "The Stability of Overconsolidated Clays" Unpublished Thesis Submitted to Inst. of Civil Eng.

8. Henkel, D.J. (1959): "The Relationship Between Strength, Pore Water Pressure, and Volume Change Characteristics of Saturated Clays" Geotechnique V. 9.

9. Henkel, D.J. (1960):"The Relationship Between Effective Stress and Water Contents of Saturated Clays" Geotechnique V. 10.

10. Henkel, D.J. (1960): "The Shear Strength of Saturated Remolded Clays" ASCE Res. Conf. on Shear Strength of Cohesive Soils; Boulder, Colo.

11. Johnson, R.W. (1962): "Physical Characteristics of Sand-Soil Mixtures under Repeated Dynamic Loads" Ph.D. Thesis, Purdue Univ.

12. Kawagami, F. and Ogawa, S. (1965): "Strength and Deformation of Compacted Soil Subjected to Repeated Stress Applications" 6th ICSM and FE V. 1:264.

13. Kersten, M.S. (1943): "Repeated Load Tests on Highway Subgrade Soil and Bases" Engineering Experiment Station Technical Paper 43, Univ. of Minn. Inst. of Tech. ; Minneapolis.

14. Knight, K. and Blight, G.E. (1965): "Studies of Some Effects Resulting from the Unloading of Soils" 6th ICSM and FE V. 11:103.

15. Larew, H.G. (1960): "Strength and Deformation Characteristics of Compacted Soil under the Action of Repeated Axial Loads" Ph.D. Thesis, Purdue Univ.

16. Lo, K.Y. (1961): "Stress-Strain Relationship and Pore water Pressure Characteristics of a Normally Consolidated Clay" 5th ICSM and FE V. 1:29.

17. Sangrey, D.A. (1968): "The Behaviour of Soils Subjected to Repeated Loading", Ph.D. Thesis Cornell Univ.

18. Seed, H.B. and Chan, C.K. (1957): "Thixotropic Characteristics of Compacted Clays" Proc. ASCE V. 83 SM4.

19. Seed, H.B. and Chan, C.K. (1958):"Effect of Stresses History and Frequency of Stress Application on Deformation of Clay Subgrades Under Repeated Loading", Proc. Hwy. Res. Bd. 38.

20. Seed, H.B. and Chan, C.K. (1961):"Effect of Duration of Stress, Application on Soil Deformation under Repeated Loading" 5th ICSM and FE V. 1:341.

21. Seed, H.B. and Chan, C.K. (1966): "Clay Strength under Earthquake Loading Conditions" Proc. ASCE V. 92 SM2.

22. Seed, H.B. and Fead, J.W.N. (1959):"Apparatus for Repeated Load Tests on Soils" ASTM Spec. Pub.No. 254.

23. Seed, H.B. and Lee, K.L. (1966):"Liquification of Saturated Sands During Cyclic Loading" Proc. ASCE V. 92 SM6.

24. Seed, H.B. and McNeill, R.W. (1957): "Soil Deformations under Repeated Stress Applications" Conf. on Soils for Engineering Purposes, Mexico City; ASTM Spec.

25. Seed, H.B.; Chan, C.K. and Monismith, C.L. (1955) "Effects of Repeated Loading on the Strength and Deformation of Compacted Clays" Proc. Hwy. Res. Bd. 34 541.

26. Seed, H.B.; McNeill, R.L. and Guenin, J. de (1958) "Increased Resistance to Deformation of Clay Caused by Repeated Loading" Proc. ASCE V. 34 SM2.

27. Seed, H.B.; Mitchell, J.K. and Chan, C.K. (1960):"The Strength of Compacted Cohesive Soils" ASCE Res. Conf. on the Shear Strength of Cohesive Soils; Boulder, Colo.

28. Tschebotarioff, G.P. and McAlpin, G.W.(1947): "The Effects of Vibratory and Slow Repetitional Forces on the Bearing Properties of Soils" U.S. Dept. of Commerce Civ. Aero. Admin. Tech. Develop. Rpt. No. 57.

29. Whitman, R.V. (1957): "The Behavior of Soils under Transient Loadings" 4th ICSM and FE V. 1:207.

30. Wroth, C.P. and Loudon, P.A. (1967):"The Correlation of Strains within a Family of Triaxial Tests on Over-consolidated Samples of Kaolin" ~~To be~~ published, Oslo, Conf.

31. Hui, T.W. ( ): The Effect of Repeated Loading on Stress-Strain Relationship of Saturated Remoulded Kaolin. 2ND. S.E. Asian Conference On Soil Engg.